# Coal Combustion Waste Impoundment Task 3- Dam Assessment Report

E. W. Brown Plant

Main Pond Dam

KENTUCKY UTILITIES

Harrodsburg, KY

### Project # 0-381

Assessment of Dam Safety
Coal Combustion Surface Impoundments
For the REAC Program

### Prepared for:

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#### INTRODUCTION

The release of over five million cubic yards of coal ash from the Tennessee Valley Authority's Kingston, Tennessee, facility in December 2008, which flooded more than 300 acres of land, damaging homes and property, is a wake-up call for diligence on coal combustion waste disposal units. The government and utilities must marshal best efforts to prevent such catastrophic failure and damage. A first step toward this goal is to assess the stability and functionality of the ash impoundments and other units, then quickly take any needed corrective measures.

This assessment of the stability and functionality of the E. W. Brown Main Pond Dam management unit is based on a review of available documents and on the site assessment conducted by Dewberry personnel on Tuesday, October 20, 2009. Dewberry found the supporting technical documentation adequate (Section 1.1.3). As detailed in Section 1.2.6, there are recommendations that may help to maintain a safe and trouble-free operation; Dewberry recommends an updated dam break analysis (currently in progress).

In summary, the E. W. Brown Main Pond Dam is SATISFACTORY for continued safe and reliable operation, with no recognized existing or potential management unit safety deficiencies.

The assessment of E. W. Brown Auxiliary Pond Dam is presented in a separate report.

#### PURPOSE AND SCOPE

The U.S. Environmental Protection Agency (EPA) is embarking on an initiative to investigate the potential for catastrophic failure of Coal Combustion Surface Impoundments (i.e., management unit) from occurring at electric utilities in an effort to protect lives and property from the consequences of a dam failure or the improper release of impounded slurry. The EPA initiative is intended to identify conditions that may adversely affect the structural stability and functionality of a management unit and its appurtenant structures (if present); to note the extent of deterioration (if present), status of maintenance and/or a need for immediate repair; to evaluate conformity with current design and construction practices; and to determine the hazard potential classification for units not currently classified by the management unit owner or by a state or federal agency. The initiative will address management units that are classified as having a Less-than-Low, Low, Significant or High Hazard Potential ranking. (For Classification, see pp. 3-8 of the 2004 Federal Guidelines for Dam Safety)

In February 2009, the EPA sent letters to coal-fired electric utilities seeking information on the safety of surface impoundments and similar facilities that receive liquid-borne material that store or dispose of coal combustion waste. This letter was issued under the authority of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) Section 104(e), to assist the Agency in assessing the structural stability and functionality of such management units, including which facilities should be visited to perform a safety assessment of the berms, dikes, and dams used in the construction of these impoundments.

EPA requested that utility companies identify all management units including surface impoundments or similar diked or bermed management units or management units designated as landfills that receive liquid-borne material used for the storage or disposal of residuals or by-products from the combustion of coal, including, but not limited to, fly ash, bottom ash, boiler slag, or flue gas emission control residuals. Utility companies provided information on the size, design, age and the amount of material placed in the units so that EPA could gauge which management units had or potentially could rank as having High Hazard Potential. The USEPA and its contractors used the following definitions for this study:

"Surface Impoundment or impoundment means a facility or part of a facility which is a natural topographic depression, man-made excavation, or diked area formed primarily of earthen materials (although it may be lined with man-made materials), which is designed to hold an accumulation of liquid wastes or wastes containing free liquids, and which is not an injection well. Examples of surface impoundments are holding, storage, settling, and aeration pits, ponds, and lagoons."

For this study, the earthen materials could include coal combustion residuals. EPA did not provide an exclusion for small units or based on whether the placement was temporary or permanent. Furthermore, the study covers not only waste units designated as surface impoundments, but also other units designated as landfills which receive free liquids.

EPA is addressing any land-based units that receive fly ash, bottom ash, boiler slag, or flue gas emission control wastes along with free liquids. If the landfill is receiving coal combustion wastes with liquids limited to that for proper compaction, then there should not be free liquids present and EPA did not seek information on such units which are appropriately designated a landfill.

In some cases coal combustion wastes are separated from the water, and the water containing de minimus levels of fly ash, bottom ash, slag, or flue gas emission control wastes, are sent to an impoundment. EPA is including such impoundments in this study, because chemicals of concern may have leached from the solid coal combustion wastes into the waste waters, and suspended solids from the coal combustion wastes remain.

The purpose of this report is to evaluate the condition and potential of waste release from the selected High Hazard Potential management units. This evaluation included a site visit. Prior to conducting the site visit, a two-person team reviewed the information submitted to EPA, reviewed any relevant publicly available information from state or federal agencies regarding the unit hazard potential classification (if any) and accepted information provided via telephone communication with a management unit supervisor.

EPA sent two professional engineers, one licensed in the State of Kentucky, for a one-day site visit. The two-person team met with the owner of the management unit as well as several technical representatives and management unit supervisors to discuss the engineering characteristics of the unit as part of the site visit. During the site visit the team collected additional information about the management unit to be used in determining the hazard potential classification of the unit. Subsequent to the site visit the management unit owner provided additional engineering data.

Factors considered in determining the hazard potential classification of the management units(s) included the age and size of the impoundment, the quantity of coal combustion residuals or by-products that were stored or disposed of in these impoundments, its past operating history, and its geographic location relative to down gradient population centers and/or sensitive environmental systems.

This report presents the opinion of the assessment team as to the potential of catastrophic failure and reports on the condition of the management unit(s). The team considered criteria in evaluating dams under the National Inventory of Dams in making these determinations.



### **LIMITATIONS**

The assessment of dam safety reported herein is based on field observations and review of readily available information provided by the owner/operator of the subject coal combustion waste management unit(s). Qualified Dewberry engineering personnel performed the field observations and review and made the assessment in conformance with the required scope of work and in accordance with reasonable and acceptable engineering practices. No other warranty, either written or implied, is made with regard to our assessment of dam safety.

### **TABLE OF CONTENTS**

INTROI	DUCTIO	N	ii
PURPO	ISE ANI	) SCOPE	ii
APPEN	DICES.		ix
1.0	CON	CLUSIONS AND RECOMMENDATIONS	1-1
1.1	CON	CLUSIONS	1-1
	1.1.1	Conclusions Regarding the Structural Soundness of the Management Unit(s)	1-1
	1.1.2	Conclusions Regarding the Hydrologic/Hydraulic Safety of the Management Unit(s)	1-1
	1.1.3	Conclusions Regarding the Adequacy of Supporting Technical Documentation	1-1
	1.1.4	Conclusions Regarding the Description of the Management Unit(s)	1-1
	1.1.5	Conclusions Regarding the Field Observations	1-2
	1.1.6	Conclusions Regarding the Adequacy of Maintenance and Methods of Operation	1-2
	1.1.7	Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program	1-2
	1.1.8	Classification Regarding Suitability for Continued Safe and Reliable Operation	1-2
1.2	REC	OMMENDATIONS	1-3
	1.2.1	Recommendations Regarding the Structural Stability	1-3
	1.2.2	Recommendations Regarding the Hydrologic/Hydraulic Safety	1-3
	1.2.3	Recommendations Regarding the Supporting Technical Documentation	1-3
	1.2.4	Recommendations Regarding the Description of the Management Unit(s)	1-3
	1.2.5	Recommendations Regarding the Field Observations	1-3
	1.2.6	Recommendations Regarding the Maintenance and Methods of Operation	1-3
	1.2.7	Recommendations Regarding the Surveillance and Monitoring Program	1-3
	1.2.8	Recommendations Regarding Continued Safe and Reliable Operation	1-3
1.3	PAR	TICIPANTS AND ACKNOWLEDGEMENT	1-4
	1.3.2	Acknowledgement and Signature	1-4
2.0		CRIPTION OF THE COAL COMBUSTION WASTE MANAGEMENT UNIT(S)	
2.1		ATION AND GENERAL DESCRIPTION	
2.2		IZE AND HAZARD CLASSIFICATION	
2.3	A	MOUNT AND TYPE OF RESIDUALS CURRENTLY CONTAINED IN THE UNIT(S) AND MAXIMUM CAPACITY	2-2

2.4	PRINCIPAL PROJECT STRUCTURES	2-3
2.4	i.1 Earth Embankment Dam	2-3
2.4	2 Outlet Structures	2-4
2.5	CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT	2-4
3.0	SUMMARY OF RELEVANT REPORTS, PERMITS AND INCIDENTS	3-1
3.1	SUMMARY OF REPORTS ON THE SAFETY OF THE MANAGEMENT UNIT(S)(S)	3-1
3.2	SUMMARY OF LOCAL, STATE AND FEDERAL ENVIRONMENTAL PERMITS	3-1
3.3	SUMMARY OF SPILL/RELEASE INCIDENTS (IF ANY)	3-1
4.0 5	SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION	4-1
4.1	SUMMARY OF CONSTRUCTION HISTORY	4-1
4.1.	.1 Original Construction	4-1
4.1.	2 Significant Changes/Modifications in Design since Original Construction	4-1
4.1.	.3 Significant Repairs/Rehabilitation since Original Construction	4-1
4.2	SUMMARY OF OPERATIONAL HISTORY	4-2
4.2	'.1 Original Operational Procedures	4-2
4.2	2.2 Significant Changes in Operational Procedures since Original Startup	4-2
4.2	2.3 Current Operational Procedures	4-2
4.2	2.4 Other Notable Events since Original Startup	4-2
5.0 F	FIELD OBSERVATIONS	5-1
5.1 F	PROJECT OVERVIEW AND SIGNIFICANT FINDINGS	5-1
5.2	EARTH EMBANKMENT DAM	5-1
5.2	2.1 Crest	5-1
5.2	2.2 Upstream Slope	5-2
5.2	2.3 Downstream Slope and Toe	5-3
5.2	2.4 Abutments and Groin Areas	5-4
5.3	OUTLET STRUCTURES	5-5
5.3	3.1 Primary Spillway	5-5
5.3	3.2 Secondary Spillway	5-6

6.0	HYDROLOGIC/HYDRAULIC SAFETY	6-1
6.1	SUPPORTING TECHNICAL DOCUMENTATION	6-1
I	6.1.1 Floods of Record	6-1
I	6.1.2 Inflow Design Flood	6-1
ſ	6.1.3 Spillway Rating	6-1
I	6.1.4 Downstream Flood Analysis	6-1
6.2	ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION	6-1
6.3	ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY	6-2
7.0	STRUCTURAL STABILITY	7-1
7.1	SUPPORTING TECHNICAL DOCUMENTATION	7-1
•	7.1.1 Stability Analyses and Load Cases Analyzed	7-1
•	7.1.2 Design Properties and Parameters of Materials	7-1
•	7.1.3 Uplift and/or Phreatic Surface Assumptions	7-1
	7.1.4 Factors of Safety and Base Stresses	7-2
	7.1.5 Liquefaction Potential	7-2
•	7.1.6 Critical Geological Conditions and Seismicity	7-3
7.2	ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION	7-4
7.3	ASSESSMENT OF STRUCTURAL STABILITY	7-4
8.0	ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION	8-1
8.1	OPERATIONAL PROCEDURES	8-1
8.2	MAINTENANCE OF THE DAM AND PROJECT FACILITIES	8-1
8.3	ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATION	8-1
}	8.3.1 Adequacy of Operational Procedures	8-1
}	B.3.2 Adequacy of Maintenance	8-1
9.0	ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM	9-1
9.1	SURVEILLANCE PROCEDURES	9-1
ļ	9.1.1 Surveillance Inspections	9-1
ļ	9.1.2 Annual Inspections	9-1
9.2	INSTRUMENTATION MONITORING	9-1
9.3	ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM	9-1
!	9.3.1 Adequacy of Inspection Program	9-1

#### **APPENDICES**

#### APPENDIX A - REFERENCE DOCUMENTS

Doc O1: E.W. Brown Ash Pond Aerial Photo, September 2009

Doc O2: FMSM Engineers Design Report, Main Ash Pond Expansion, October 2007

Doc 03 - 87: Main Ash Pond Expansion Construction Drawings, October 2007, FMSM Engineers

Doc 88: Kentucky Division of Water Dam Inspection Report, July 30, 2008

Doc 89: ATC Associates Dam Inspection Report, January 2009

Doc 90: Embankment Cross Sections Station 228+00 - Station 231+00, Drawing 31/71, Revised November 19,

1991, FMSM Engineers.

### APPENDIX B - PHOTOGRAPHS

Photographs 1 - 47

#### APPENDIX C - FIELD OBSERVATION CHECKLIST

Dam Inspection Checklist Form

#### 1.0 CONCLUSIONS AND RECOMMENDATIONS

#### 1.1 CONCLUSIONS

Conclusions are based on visual observations from a one-day site visit and review of technical documentation provided by E.ON U.S. LLC.

1.1.1 Conclusions Regarding the Structural Soundness of the Management Unit(s)

Based on a review of the engineering data provided by the owner's technical staff and Dewberry's observations during the site visit, the embankment appears to be structurally sound.

The Main Pond had been taken out of service prior to the site observations. Construction was underway as part of the planned phased expansion of the facility. The Main Pond had been dewatered and the emergency spillway abandoned using procedures prescribed by the design engineer of record.

The owner provided data included information pertaining to liquefaction potential, slope stability and hydrologic/hydraulic characteristic of the expanded and reconfigured Main Pond. Dewberry assumes that the Kentucky Division of Water conducted an appropriate full review prior to issuing a construction permit.

1.1.2 Conclusions Regarding the Hydrologic/Hydraulic Safety of the Management Unit(s)

The E. W. Brown Main Pond has been dewatered and taken out of service. The emergency spillway has been abandoned. The primary spillway remains but will be abandoned as part of the facility expansion. A new primary spillway is under construction at an alternate location within the footprint of the reconfigured Main Pond.

1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation

Supporting technical documentation is adequate. Although documentation of the existing embankment is somewhat limited, the design documentation for the Main Pond incorporates prior data and presents stability analyses that incorporate a review of the existing dam.

1.1.4 Conclusions Regarding the Description of the Management Unit(s)

The description of the management unit provided E.ON U.S. LLC was an accurate representation of what Dewberry engineers observed in the field.

#### 1.1.5 Conclusions Regarding the Field Observations

Dewberry engineers were provided access to all areas in the vicinity of the management unit required to conduct a thorough field observation. The visible parts of the embankment dam were observed to have no signs of overstress, significant settlement, shear failure, or other signs of instability. The embankment dam visually appears structurally sound. There are no apparent indications of unsafe conditions or conditions needing remedial action.

1.1.6 Conclusions Regarding the Adequacy of Maintenance and Methods of Operation

The current maintenance and methods of operation appear to be adequate for the management unit. There was no evidence of repaired embankments or prior releases observed during the site visit.

1.1.7 Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program

Surveillance and monitoring program appear to have been adequate. A new surveillance and monitoring program is planned for implementation when the reconfigured Main Pond is put back into service.

1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation

The E. W. Brown Main Pond facility is currently out of operation and important components, including the emergency spillway, have been abandoned using procedures prescribed by the design engineer of record. The embankment is considered stable at this time.

Analyses conducted in conjunction with the expansion and reconfiguration of the Main Pond indicate that the existing ash, on which the existing north embankment is constructed, is subject to liquefaction if groundwater elevation is above 856 feet. Groundwater elevation at the start of the current phase of construction was 870 feet. The expansion plan anticipates that groundwater elevations will recede while the pond is out of service and continue to recede once the Phase I construction pond liner is installed. Groundwater elevations will be monitored during the Phase I construction and during the interim between Phase I and Phase 2, expected to be about one year. If groundwater does not recede to elevation 856 or lower, a drainage system will be installed in the fly ash to control groundwater to an elevation of 856 or lower.

Upon completion of the current expansion phase, the facility will have a substantially different configuration.

#### 1.2 RECOMMENDATIONS

1.2.1 Recommendations Regarding the Structural Stability

No recommendations regarding structural stability appear warranted at this time.

1.2.2 Recommendations Regarding the Hydrologic/Hydraulic Safety

No recommendations appear warranted at this time.

1.2.3 Recommendations Regarding the Supporting Technical Documentation

No recommendations appear warranted at this time.

1.2.4 Recommendations Regarding the Description of the Management Unit(s)

No recommendations appear warranted at this time.

1.2.5 Recommendations Regarding the Field Observations

No recommendations appear warranted at this time.

1.2.6 Recommendations Regarding the Maintenance and Methods of Operation

The maintenance and operation of the dam appear to have been adequate. However, updating the 1991 Operations Plan should be completed prior to reopening the reconfigured Main Pond at the completion of the current phase of construction.

1.2.7 Recommendations Regarding the Surveillance and Monitoring Program

No recommendations pertaining to the surveillance and monitoring program appear warranted at this time.

1.2.8 Recommendations Regarding Continued Safe and Reliable Operation

No recommendations pertaining to the continued safe and reliable operation of the management unit appear warranted at this time.

#### 1.3 PARTICIPANTS AND ACKNOWLEDGEMENT

### 1.3.1 List of Participants

W. Michael Winkler – E.DN U.S. LLC Jeffrey B. Heun, P.E. – E.DN U.S. LLC David J. Millay, P.E. – E.DN U.S. LLC Jeffrey Fraley – KU Tamara Lay – KU Hugh A. Ward, P.E. – Dewberry Joseph P. Klein, III, P.E. – Dewberry

### 1.3.2 Acknowledgement and Signature

We a	cknowledge that	the management u	nit reference	d herein ha	s been	assessed o	n October	20,	2009.
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Hugh A. Ward

Professional Engineer

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Secretary-Treasurer

Hugh & Ward, PE (KY # 7164)

Joseph P. Klein, III, P.E. Geotechnical Engineer

#### 2.0 DESCRIPTION OF THE COAL COMBUSTION WASTE MANAGEMENT UNIT(S)

#### 2.1 LOCATION AND GENERAL DESCRIPTION

The E. W Brown Plant is located near the west bank of the Dix River, just upstream of Dix Dam at Herrington Lake in Mercer County, Kentucky approximately 5 miles northeast of Burgin, Kentucky. The plant is operated by Kentucky Utilities Company, an operating company of E.ON U.S. LLC (E.ON). The Main Pond Dam is at the southwest side of the plant site, adjacent to the Auxiliary Pond. A project location aerial photograph is provided in Appendix A – Doc O1.

The E. W. Brown existing Main Pond Dam is a compacted clay embankment with zones of graded stone filters and shot rock drains. The pond is not lined. The crest of the dam is at elevation 900 feet. The downstream toe of the dam is at elevation 774 feet, making the dam height 126 feet.

Construction has begun on the first phase of a multi-phased expansion of the Main Pond. Phase I construction consists of a new dike constructed upstream from the existing dam with a center line approximately 400 feet upstream from the existing dam. The new dike, referred to as the "starter dike" on construction drawings, has a design crest elevation of 902 feet, 2 feet higher than the existing dam. When the Main Pond is put back into service all storage is designed to be upstream of the new dike. Planned future phases of expansion will raise the crest of the starter dike by increasing width downstream toward the existing dam. The starter dike and planned subsequent expansions are supported on existing de-watered and stabilized ash materials.

#### 2.2 SIZE AND HAZARD CLASSIFICATION

The existing Main Pond Dam is on the southwest side of the E. W. Brown generating station. The existing dam has a maximum height of 126 feet and impounds approximately 126 acres (see Table 2.3-1 and Table 2.4-1). The dam crest length is 2,175 feet and the dam crest width is 20 feet. The dam crest elevation is at 900 feet and elevation at the lowest downstream toe of the dam is 774 feet.

The classification for size, based on the height of the dam, is "Large" with the USACE Recommended Guidelines for Safety Inspection of Dams ER 1110-2-106 criteria summarized in Table 2.2a.

Table 2.2a USACE ER 1110-2-106 Size Classification		
	Impoundment	
Category	Storage (Ac-ft)	Height (ft)
Small	50 and < 1,000	25 and < 40
Intermediate	1,000 and < 50,000	40 and < 100
Large	> 50,000	> 100

The E. W. Brown Main Pond dam is classified by the Kentucky Department of Environmental Control Division of Water (KYDW) as Class C – High Hazard Structure. The KYDW rules define High Hazard structures as: ".....structures located such that failure may cause loss of life, or serious damage to houses, industrial or commercial buildings, important public utilities, main highways or major railroads. This classification must be

used if failure would cause probable loss of human life." This classification definition is similar to "High" classification per the Federal Guidelines for Dam Safety dated April 2004. As shown in Table 2.2b, dams assigned the "high hazard potential?" classification are those dams where failure or error of operation results in the probable loss of one or more human life is expected, probable economic loss, environmental damages and disruption of lifeline facilities.

Table 2.2b FEMA Federal Guidelines for Dam Safety Hazard Classification					
Hazard Potential	Hazard Potential				
Classification	Loss of Human Life	Economic, Environmental, Lifeline Losses			
Low	None Expected	Low and generally limited to owner			
Significant	None Expected	Yes			
High	Probable. One or more expected	Yes (but not necessary for this classification)			

#### 2.3 AMOUNT AND TYPE OF RESIDUALS CURRENTLY CONTAINED IN THE UNIT(S) AND MAXIMUM CAPACITY

The Main Pond is designed to manage fly ash, bottom ash, flue gas desulphurization residuals, pyrites, and other process waters. The data reviewed by Dewberry included Design Report dated October 19, 2007 (see Appendix A: Doc. 02) prepared by Fuller, Mossbarger, Scott & May Engineers, Inc. Data on the volume of residuals stored in the Main Pond at the time of inspection were not indicated. The surface area for the pond at normal pool elevation is approximately 126. The current volume of ash stored in the Main Pond was not provided. The total ash storage capacity for each phase of expansion is provided in the Table 2.3-2.

Table 2.3-1: Amount of Residuals and Maximum Capacity of Unit				
E. W. Brown Main Pond				
Surface Area (acre)	126			
Current Storage Capacity (acre-feet)	Data not provided			
Total Storage Capacity (acre-feet)	See Table 2.3-2			
Crest Elevation (feet)	900			
Normal Pond Level (feet)	893			

The existing Main Pond has been taken out of service. When the reconfigured pond is put back in service the area between the existing main pond and the starter dike will not be a part of the storage basin.

Subsequent phases of expansion will incrementally raise the crest elevation of the new dike to a final elevation of 962 feet. Raising the crest elevation will be accomplished by broadening the base in the downstream direction, filling in the unused space between the new dike and the existing dam. A schematic of the proposed expansion phases is provided on Figure 3 incorporated into the Design Report (Appendix A: Doc 02). The total storage capacity of the Main Pond for each phase of the expansion project is:

Table 2.3-2: Storage Capacity of Reconfigured Main Pond for Each Phase						
	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	
Surface Area (acre)	73.45	80.14	88.50	97.87	106.42	
Storage Capacity (acre-feet)	868	1655	3062	4740	6350	
Dam Crest Elev. (feet)	902.0	912.0	928.0	946.0	962.0	
Normal Pond Level (feet)	897.55	907.90	924.40	942.40	958.16	

#### 2.4 PRINCIPAL PROJECT STRUCTURES

#### 2.4.1 Earth Embankment Dam

The existing Main Pond Dam is a soil and rock fill dam constructed in three stages. The initial dam was constructed prior to the 1970s. The initial crest elevation was approximately 830 feet. The dam was expanded in the 1970s to a crest elevation of 870 feet and in the early 1990s to the current crest elevation of 900 feet. The original dam is reportedly supported on rock, although the expansions generally consisted of widening the dam in the downstream direction, drawings for the current expansion program indicate that the upstream toe of the 1970s expansion is located partially over ash. (See Appendix A; Doc 57 and 58). Table 2.4.1-1 displays a summary of dimensions and size specifications for the E. W. Brown Main Pond Dam. Photo Numbers 1 – 9, 11 – 17, 25 – 27, 30, 37 – 39, 44, and 45 show the embankment of the dam.

Table 2.4.1-1: Summary of Dam Dimensions and Size			
E. W. Brown Main Pond Dam			
Dam Height	126'		
Crest Width	20'		
Length	2,175'		
Side Slopes (upstream)	2.5:1		
Side Slopes (downstream)	2.5:1		
Hazard Classification Class C – High Hazard			

<sup>&</sup>quot;As constructed" embankment cross-sections of the Main Pond Dam 1990 expansion indicate sections of a 6 foot deep cut-off trench were added to sections of the new dam..

#### 2.4.2 Outlet Structures

The existing Main Pond had a principal spillway and an emergency spillway. Since the facility has been taken out of service and dewatered, the emergency spillway had been abandoned using procedures prescribed by the design engineer of record. As the principal spillway is located in an area that will not receive sluiced coal combustion waste, it is scheduled to be grouted and abandoned.

Construction drawings show that the area of the existing pond between the existing dam with a crest elevation of 900 feet and the new starter dike with a crest elevation of 902 feet will not be used for ash storage. The area is to be graded to provide positive drainage to a surface water storm drainage system (see Appendix A: Doc 24).

#### 2.5 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT

A dam break analysis, including the identification of critical infrastructure located within 5 miles downstream of the dam is currently underway.

Based on observations at the site and surrounding area, the critical infrastructure includes the railroad line serving the E. W. Brown generating station, the Dix Dam and local roadways. Also at risk are residences along the bank of Herrington Lake in the vicinity of the plant.



#### 3.0 SUMMARY OF RELEVANT REPORTS, PERMITS AND INCIDENTS

#### 3.1 SUMMARY OF REPORTS ON THE SAFETY OF THE MANAGEMENT UNIT(S)

In response to a Freedom of Information request, E.ON U.S. LLC provided an extensive package of design information, performance monitoring data and past inspection documents for the E.W. Brown Main Pond Dam. The data were provided in the form of electronic files that are included in Appendix A. Reports directly relevant to the safety of the Main Pond Dam are summarized below.

The Kentucky Division of Water inspected the Main Pond Dam on July 30, 2008 (Appendix A: Doc 88). The report indicates no signs of slides, slumps or cracking on either the downstream or upstream slopes of the embankment. The report also indicates no signs of cracking or subsidence on the crest of the dam. The next Kentucky Division of Water inspection is scheduled for 2010.

KU retained ATC Associates, Inc to conduct an inspection of the existing Main Pond Dam in 2009. The ATC Associates inspection was conducted on January 11, 2009 and reported the dam to be in generally good condition (Appendix A: Doc 89). The inspection reported issues at two general areas of the existing dam:

•	Crest
washout area under sprinkler line	
0	Small
depression where drawdown pipe trenc	h was backfilled
0	Two
irregularities in width of crest on upstr	eam slope of east embankment
•	Seepage:
0	Minor
amount of seepage at the north abutme	nt
0	
at toe of east slope	

Recommendations for repairs were provided with priority ratings of "moderate" and "normal."

#### 3.2 SUMMARY OF LOCAL, STATE AND FEDERAL ENVIRONMENTAL PERMITS

The Kentucky Division of Water has assigned Dam ID Number KYDW Permit 0737 to this structure. Kentucky inspects the dam on a biennial basis. The dam was inspected by the Kentucky Division of Water in 2008 and is scheduled for another State inspection in 2010.

The E. W. Brown Main Pond spillway discharge is permitted under KPDES Permit No. 0002020 which expired January 31, 2007. The permit remains in effect under applicable state regulations. A renewal application was submitted in mid-2006. A new permit has been issued and will be effective on March 1, 2010.

3.3 SUMMARY OF SPILL/RELEASE INCIDENTS (IF ANY)

Data included in the review documentation did not indicate any spills, unpermitted release, or other performance related problems with the dam over the last 10 years.



#### 4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION

#### 4.1 SUMMARY OF CONSTRUCTION HISTORY

### 4.1.1 Original Construction

The reviewed documents did not include the original design and construction records. However, it is understood that initial construction of the Main Pond was prior to 1970. The dam was expanded in the 1970s and again in the early 1990s to the current crest elevation of 900 feet. Documentation provided for review indicates the existing dam is primarily a compacted clay embankment with additional zones of graded stone filters and shot rock drains (Appendix A: Doc 33). Available drawings indicate a shallow cutoff wall beneath a segment of the existing dam (Appendix A: Doc 90).

Drawings summarizing the results of stability analyses for the expansion and reconfiguration of the Main Pond dam include a schematic representation of the existing dam. The schematic drawing indicates the dam was constructed in three phases:

•		Original
	Embankment with a crest elevation of approximately 830 feet.	J
•		1970's
	Embankment with a crest elevation of approximately 870 feet.	
•		1990's
	Embankment with a crest elevation of 900 feet.	

### 4.1.2 Significant Changes/Modifications in Design since Original Construction

According to the information included in the design report in Appendix A: Doc O2, the Main Pond was expanded multiple times through the early 1990s. Construction is currently underway to expand and reconfigure the facility. A new dike is being constructed about 400 feet upstream of the existing dam such that the area between the starter dike and existing dam will no longer be part of the storage basin. The area will be the base of planned future expansion of the starter dike from an initial crest elevation of 902 feet to a final crest elevation of 962 feet.

The starter dike as well as subsequent planed phases of expansion is supported on dewatered and stabilized fly ash in the pond. Liquefaction analyses in the Design Report (See Appendix A: Doc 2) indicate a potential for liquefaction of the ash under the existing north embankment if groundwater is above elevation 856 feet. Groundwater elevation at the time of the design was 870 feet. The design analyses assumed with the pond out of service, and installation of a new pond liner should cause groundwater to recede. Current construction includes installation of monitoring wells beneath the existing north dike to monitor groundwater elevation between the current construction and Phase 2 construction, expected to commence in 2011. If the groundwater elevation has not dropped below elevation 856 or lower, a drainage system will be installed to lower the groundwater elevation and stabilize the existing embankment against a potential liquefaction failure.

4.1.3 Significant Repairs/Rehabilitation since Original Construction

No information was provided regarding major repairs or rehabilitation of the existing dam. No evidence of prior releases, failures, or patchwork was observed on the earthen embankment during the visual site assessment and no documents or statements were provided to the dam assessor that indicates prior failures have occurred.

#### 4.2 SUMMARY OF OPERATIONAL HISTORY

#### 4.2.1 Original Operational Procedures

The reviewed documentation did not include the original operation procedures. The Main Pond has been operated under procedures developed in 1991 after the last expansion. The facility is currently out of service and undergoing reconfiguration and expansion. New operating procedures, including an Emergency Action Plan, are being developed for the reconfigured impoundment.

#### 4.2.2 Significant Changes in Operational Procedures since Original Startup

No documents have been provided to indicate any operational procedures have changed. However the current construction to expand and reconfigure the impoundment (see Section 4.1.2) implies a change in operating procedures.

#### 4.2.3 Current Operational Procedures

The Main Pond is currently out of service. Coal combustion waste material is currently being sent to the Auxiliary Pond during the ongoing expansion and reconfiguration of the Main Pond.

### 4.2.4 Other Notable Events since Original Startup

No notable events have been reported nor has the dam has experienced spills or unpermitted releases in the last 10 years.

#### 5.0 FIELD OBSERVATIONS

#### 5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS

Dewberry performed a site visit on Tuesday, October 20, 2009. The site visit began at 09:00 AM. The weather was clear and warm. Please refer to photographs in Appendix B taken by Dewberry during the October 20, 2009 dam inspection and the Dam Inspection Checklist, Appendix C. Selected photographs are included here for ease of visual reference. The overall assessment of the dam was that it was in satisfactory condition and no significant findings were noted.

#### 5.2 EARTH EMBANKMENT DAM

#### 5.2.1 Crest

The crest of the existing dam had no signs of any depressions, tension cracks or other indications of settlement or shear failure, and appeared to be in satisfactory condition. Figure 5.2.1-1 shows the crest of the existing Main Pond Dam.



Figure 5.2.1-1 Crest of Main Pond Dam Looking Southward.

### 5.2.2 Upstream Slope

The upstream slope mostly consists of unprotected compacted soil. The upstream slope mostly consists of unprotected compacted soil. Figure 5.2.2-1 shows the upstream slope of the existing embankment on the east side of the impoundment. Scarps, sloughs, bulging, cracks, scarps, depressions, or other indications of slope instability or signs of erosion were not observed. The less steep slope in the foreground of the photograph is an access ramp for construction equipment working in the out-of-service impoundment.



Figure 5.2.2-1. The Upstream Slope of the Main Dam (the Embankment on the Left Side of the Picture)

### 5.2.3 Downstream Slope and Toe

The downstream slope is protected with graded stone aggregate. Scarps, sloughs, depressions or other indications of slope instability or signs of erosion or uncontrolled seepage were not observed. Figure 5.2.3-1 shows the downstream slope at the southeastern side of the impoundment, the highest portion of the dam. Figure 5.2.3-2 shows the downstream slope along the northeastern side of the impoundment.



Figure 5.2.3-1. Downstream Slope at the Northeast Side of Impoundment

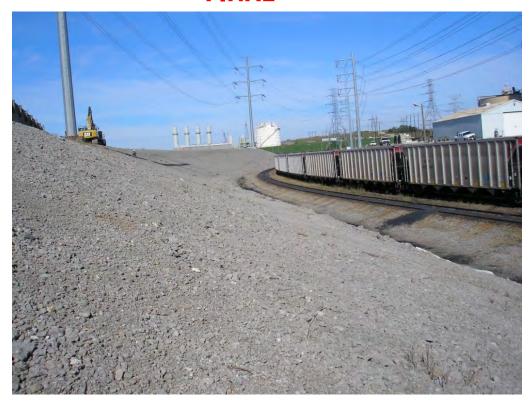


Figure 5.2.3-2. Downstream Slope at the Northeast Side of Impoundment

### 5.2.4 Abutments and Groin Areas

The abutments and groin areas appeared to be in good condition.

### 5.3 OUTLET STRUCTURES

### 5.3.1 Primary Spillway

The existing primary spillway consists of a vertical decant riser and a 24-inch diameter corrugated metal discharge pipe (Figure 5.3.1-1). As the pond is currently out of service no water was flowing through the primary spillway at the time of Dewberry's inspection. The primary riser is scheduled to be grouted and abandoned as part of the current expansion and reconfiguration construction.



Figure 5.3.1-1. Existing Primary Spillway Structure.

A new primary spillway was under construction at the time of Dewberry's site visit.

### 5.3.2 Secondary Spillway

The existing emergency spillway has been abandoned using procedures prescribed by the design engineer of record. Figure 5.3.2-1 shows a new secondary spillway discharge end at the Auxiliary Pond.



Figure 5.3.2-1. Secondary Spillway from Reconfigured Main Pond to Auxiliary Pond (Discharge End Shown)

#### 6.0 HYDROLOGIC/HYDRAULIC SAFETY

### **6.1 SUPPORTING TECHNICAL DOCUMENTATION**

#### 6.1.1 Floods of Record

No documentation has been provided about the floods of record.

#### 6.1.2 Inflow Design Flood

A calculation of the inflow design flood used for the existing pond was not included in the reviewed documents. The pond is now out of service. The reconfigured facility currently under construction includes a new upstream embankment with a crest elevation 2 ft. higher than the existing dam. When the pond is reopened in its new configuration, the area in which the existing spillways are located will not be within the water storage footprint.

Data reviewed for the new configuration indicates that the new upstream embankment will handle the PMP event without overtopping.

#### 6.1.3 Spillway Rating

The spillway rating for the existing spillway was not found in the reviewed data. As the facility is out of service during construction of a reconfigured impoundment, the primary spillway is out of service and scheduled to be abandoned before the facility is reopened.

The existing emergency spillway has been abandoned using procedures prescribed by the design engineer or record.

Hydraulic and hydrologic data provided for the expanded and reconfigured Main Pond indicates that both the starter dike and final configuration can pass the PMP without overtopping. The data indicates the starter dike freeboard at the Probable Maximum Precipitation (PMP) is 1.4 feet and at the final embankment configuration freeboard is 1.5 feet (see Appendix A: Doc. 43).

#### 6.1.4 Downstream Flood Analysis

A downstream flood analysis was not performed as part of the E, W. Brown Main Pond dam design. A dam break analysis is currently being conducted, but results were not available at the time of Dewberry's evaluation.

#### 6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Supporting technical documentation is inadequate to assess the original facility, but the design for the 1990 main pond extension included a geotechnical exploration program that evaluated the existing dam and whose findings were incorporated into the design of the 1990 extension. Most of the provided information addressed the dam's expansion.

#### 6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY

The original hydrology/hydraulic assessment used for the design of the Main Pond was not included in the reviewed documents. However, according to E. ON U.S. LLC a dam break analysis for the Main Pond was completed in November 2009, and incorporated into an Impoundment Emergency Action Plan for the Main Pond in January 2010.

Note: the facility is out of service and no new coal combustion waste material is being added to the impoundment.

The reconfigured facility includes a new primary spillway and new secondary spillway. The new primary spillway, just beginning construction at the time of this assessment, will be a vertical decant riser with an invert elevation of 895 feet. The primary spillway will connect to the existing outfall system

The new secondary spillway consists of a 30-inch diameter HDPE pipe with an invert elevation of 892.5 feet. The secondary spillway discharges into the adjacent Auxiliary Ash Pond.

Technical data provided is adequate to assess the new design Main Pond configuration.

#### 7.0 STRUCTURAL STABILITY

#### 7.1 SUPPORTING TECHNICAL DOCUMENTATION

#### 7.1.1 Stability Analyses and Load Cases Analyzed

The reviewed documents did not include the original stability analysis, design calculations or field measurements for the existing Main Pond. However, the design report for the expansion of the Main Pond currently underway includes analyses for the existing dam for both the Phase I expansion and the final expansion configurations (see Appendix A: Doc 57, 58 and 59). The analyses were conducted using UTEXAS4 software.

Stability analyses were conducted for long term stability of upstream and downstream embankments for shall and deep rotational failures. Analyses were conducted for normal pool and no pool conditions.

The stability analyses (Appendix A: Doc 02, 57, 58, and 59), for dynamic conditions were conducted using a pseudo-static loading condition based on a peak ground acceleration of 0.100g for a two percent probability of exceedance in 50 years.

#### 7.1.2 Design Properties and Parameters of Materials

The design parameters used for the original dam design were not available from the reviewed documents.

However, design parameters for the stability analysis for the reconfiguration and expansion program currently underway (see Appendix A: Doc. 57, 58, and 59) are available. These parameters at least partially reflect the properties of the existing embankment. The density values listed in the parameter tables for the downstream slope range from 110 to 118 pounds per cubic foot (PCF). Angle of shearing resistance under effective stress analysis range is 28° to 38° for various zones and, where applicable, the effective cohesive strength is 100 pound per square foot.

#### 7.1.3 Uplift and/or Phreatic Surface Assumptions

No uplift considerations are included in the stability analyses. The reconfigured Main Pond and new embankment upstream slope of the embankment are lined with a 2-foot thick clay zone capped by a 60 mil Liner Low-Density Polyethylene (LLDP) flexible membrane liner (see Appendix A: Doc. 33).

In the stability analysis section of the design report for the proposed expansion and reconfiguration of the Main Pond (see Appendix A- Doc O2) a phreatic level was shown as a horizontal surface at elevation 870 feet.

### 7.1.4 Factors of Safety and Base Stresses

The reviewed documents did not include any information about the factors of safety and base stresses for the original design of the existing embankment.

In the stability analysis section of the design report for the proposed expansion and reconfiguration of the Main Pond (see Appendix A- Doc O2) the static and pseudo-static stability safety factors for the existing embankment are shown for the downstream slope. The report indicates that the pseudo-static analysis is without liquefaction. The computed Safety Factors are listed in Table 7.1.4.

Table 7.1.4: Factors of Safety E. W. Brown Main Dam (Note 1)				
Location/Loading Condition	Required Safety Factor (Army Corps)	Computed Safety Factor		
Final Dam Configuration (Crest Elev. 962)Upstream – Long Term Shallow Failure, No Pool , Static	1.5	2.1		
Upstream – Long Term Shallow Failure, No Pool , Dynamic	1.2	1.8		
Upstream – Long Term Deep Failure, No Pool , Static	1.5	2.3		
Upstream – Long Term Deep Failure, No Pool , Dynamic	1.2	1.8		
Downstream – Long Term Shallow Failure, Static (Note 2)	1.5	2.1		
Downstream – Long Term Shallow Failure, Dynamic	1.2	1.3		
Downstream – Long Term Deep Failure, Static	1.5	2.2		
Downstream – Long Term Deep Failure, Dynamic	1.2	1.6		
Starter Dike Long Term No pool, Static	1.5	2.0		
Starter Dike Long Term No pool, Dynamic	1.2	1.4		

Notes: 1 – Results are for Main Dam in final proposed configuration with crest elevation of 962 feet 2 – Shallow failure surface is contained within existing Main Pond embankment.

#### 7.1.5 Liquefaction Potential

No liquefaction potential data were submitted for the existing embankment.

The design report for the expansion and reconfiguration of the Main Pond (see Appendix A- Doc D2) includes an evaluation of liquefaction potential for fly ash underlying the existing north embankments. The results of the evaluation indicated a potential for liquefaction in the fly ash materials in conditions resulting in a phreatic surface about elevation 856 feet. The report concludes that liquefaction could destabilize the existing dike and could cause progressive sliding of the planned larger dike. Based on the identified hazard, the design includes provisions for monitoring ground water levels beneath the starter dike for the period between Phase 1 and Phase 2 construction, expected to be about one year. If the water level does not recede as expected, a drainage system will be incorporated into the Phase 2 construction to control the groundwater lever at or below elevation 856 feet.

#### 7.1.6 Critical Geological Conditions and Seismicity

Data in the Dam Construction Permit Application (see Appendix A: Doc O2) indicate the E. W. Brown Main Pond is underlain by rock of the Lexington and Tyrone Limestone formations. Members of the Lexington formation at the site include: Greer Limestone, Logana Limestone, and Curdsville Limestone. The Tyrone Limestone formation underlies the Curdsville Limestone.

Geologic maps of Kentucky identify the carbonate rock formations at the site as susceptible to formation of sinkholes. Drawings for the current expansion construction include provisions for treating discontinuities observed in the rock surface during construction (see Appendix A Doc 32). The same rock treatment requirements were included on the 2006 construction drawings for the adjacent Auxiliary Pond Dam; however, the "as constructed" drawings do not indicate areas requiring treatment.

Drawings of the 1990 expansion of the existing Main Pond Dam indicate that isolated solution features were observed near the downstream toe of the expanded embankment. The drawings indicate that the areas were treated by backfilling surface cavities with course aggregate and a geotextile filter fabric.

The Design Report includes boring logs from several geotechnical explorations at the Main Pond. Borings at the southwest abutment and along the northern leg of the dam include rock coring data. The rock coring data indicate recoveries generally ranging from 60 to 100 percent and Rock Quality Designations (RQD) generally ranging from 24 to 85 percent. The values are consistent with the rock description of "thin bedded, irregular/nodular bedding with shale stringers and partings".

The rock core data and the filed notes on the 1990 "as constructed" drawings suggest that solution features are limited to localize cavities, and that design have included filed treatment procedures when irregularities in the rock are encountered.

The documents provided indicate that seismicity was considered in the design. The slope stability analyses included a dynamic load condition based on a peak ground acceleration of 0.100 g.

As part of this assessment the current Seismic Risk Map of the United States was also reviewed using the U. S. Geologic Survey web site. The 2%/50 year return period peak ground acceleration mapped for the sire is 0.100 g. The seismic design criteria are appropriate for this dam.

### 7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Structural stability documentation is limited for the existing Main Pond. However, there is adequate information in the design report for the expansion and reconfiguration of the Pond to assess the structural stability of the existing embankment.

### 7.3 ASSESSMENT OF STRUCTURAL STABILITY

Overall, the structural stability of the Main Pond embankment appears to be satisfactory based on the following observations during the October 20, 2009 field visit and dam evaluation by Dewberry, the 2006 Dam Construction Application Report, and the post-construction drawings.

•	Ther
were no indications of scarps, sloughs, depressions or bulging anywhere along the dam;	
•	Boils
sinks or uncontrolled seepage was not observed along the slopes, groins or toe;	
•	The
computed factors of safety comply with accepted criteria.	



#### 8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION

#### 8.1 OPERATIONAL PROCEDURES

The facility is currently out of service. The facility is to be restored to service upon completion of Phase I of a five-phase expansion and reconfiguration program. Phase I construction is currently underway.

Prior to being taken out of service, the Main Pond Dam was operated in accordance with the 1991 Operation Plan prepared in conjunction with the last expansion of the embankment. A new Operations Plan and Emergency Action Plan were recently completed (January 2010) for the expanded and reconfigured Main Pond.

Discharge from the outflow structure is to Herrington Lake. The facility KPDES permit (KY 0002020) has expired, but remains in effect under applicable state regulations. A renewal application was submitted prior to the expiration date. A new permit has been issued and will be in effect on March 1, 2010.

#### 8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES

Maintenance procedures for the Main Pond include:

- Weekly inspections by plant personnel;
- Annual engineering inspection;
- Removal of vegetation from joints, resealing and repair of joints/cracks in concrete sections as required;
- Repair of vehicle/traffic damages and replacement or repair of access gates as required.

#### 8.3 ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATION

#### 8.3.1 Adequacy of Operational Procedures

Based on the assessments of this report operation procedures seem to have been adequate.

#### 8.3.2 Adequacy of Maintenance

Various dam inspection reports including the Kentucky Division of Water inspection report of July 30, 2008 (see Appendix A: Doc 88), and the ATC Associates, Inc. report of January 22, 2009 (see Appendix A: Doc 89) reported no major maintenance issues. Although several maintenance recommendations were made, none of them are considered critical or imminent. This indicates that the maintenance plan is probably followed in practice and adequate maintenance is provided for the dam and the project facilities.

Although the maintenance program is adequate, several recommendations have been made to improve the maintenance and insure trouble-free operation.

The ATC Associates, Inc. January 22, 2009 recommended:

- Filling depression under a sprinkling line
- Repair of upstream crest narrowing
- Excavate and refill depressions at downstream slope at previous drawdown pipe location
- Install weir to allow monitoring of flow
- Monitor flow to evaluate seepage from cooling tower discharge to fly ash impoundment
- Remove blockage in Emergency Spillway prior to placing facility back in service
- Prepare Operations and Maintenance Plan for all aspects of the structure
- Prepare Emergency Operations Plan for structure distress scenarios
- Institute and document regular facility inspection plan
- Conduct visual inspection of the facility during the 2008 growing season
- Prepare current topographic mapping

The Dewberry engineering team site visit (October 20, 2009) or subsequent dam assessment did not result in any other major observations or additional maintenance recommendations to the items listed above.



#### 9.0 ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM

#### 9.1 SURVEILLANCE PROCEDURES

#### 9.1.1 Surveillance Inspections

Surveillance inspections of the Main Pond are conducted weekly. A written summary of observations is provided to facility management.

### 9.1.2 Annual Inspections

A third party inspection was conducted January 22, 2009 by ATC Associates. The inspection report identified did not identify any high priority issues. Some of the recommendations made in the ATC Associates report have been addressed by the construction of the new facility configuration; e.g., the emergency spillway has been abandoned using procedures prescribed by the design engineer of record and new primary and secondary spillways designed.

#### 9.7 INSTRUMENTATION MONITORING

The Main Pond monitoring system consisted of a contained series of piezometers. Monitoring was suspended when the impoundment was taken out of service.

A network of piezometers is included in the design of the expanded and reconfigured Main Pond

#### 9.3 ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM

#### 9.3.1 Adequacy of Inspection Program

Based on the data reviewed by Dewberry, including observations during the site visit, the inspection program is adequate.

#### 9.3.2 Adequacy of Instrumentation Monitoring Program

An instrumentation monitoring program was implemented but there is little evidence that results were being tracked and analyzed for changes in conditions that might be detrimental to the embankment.

### E. W. Brown Main Ash Pond

### Appendix A

### Table of Contents

Doc. No. 1 E. W. Brown Main Fly Ash Pond Aerial Photo 2 FMSM Engineers Design Report - Main Ash Pond Expansion , October 2001/A BRN Exp Design Rpt 2007.pdf 3 128 Cover Sheet Main Ash Pond - Starter Dike 128 COO8DO.pdf 4 129 Index of Sheets and Revisions Main Ash Pond – Starter Dike 129 C00801.pdf 5 130 General Notes Main Ash Pond – Starter Dike *130 C00802.pdf* 6 131 Plan View Project Setting Main Ash Pond – Starter Dike *131 C00803.pdf* 7 132 Plan View Map Key Main Ash Pond – Starter Dike 132 COO804.pdf 8 133 Starter Dike Overview Plan – Main Ash Pond – Starter Dike *133 C00805.pdf* 9 134 Final Embankment Overview Plan Main Ash Pond – Starter Dike 134 C00806.pdf 10 135 Existing Conditions and Baseline Layout - Map 1 Main Ash Pond - Starter Dike *135 C00807.pdf* 11 136 Existing Conditions and Baseline Layout - Map 2 Main Ash Pond - Starter Dike 136 C00808.pdf 12 137 Existing Conditions and Baseline Layout - Map 3 Main Ash Pond - Starter Dike *137 C00809.pdf* 13 138 Existing Conditions and Baseline Layout – Map 4 Main Ash Pond – Starter Dike *138 COOSIO.pdf* 14 139 Existing Conditions and Baseline Layout - Map 5 Main Ash Pond - Starter Dike 139 COO811.pdf 15 140 Existing Conditions and Baseline Layout - Map 6 Main Ash Pond - Starter Dike 140 C00812.pdf 16 141 Construction Plan Map 1 Main Ash Pond – Starter Dike 141 COO813.pdf 17 142 Construction Plan Map 2 Main Ash Pond – Starter Dike 142 COO814.pdf 18 143 Construction Plan Map 3 Main Ash Pond – Starter Dike 143 COO815.pdf 19 144 Construction Plan Map 4 Main Ash Pond – Starter Dike 144 COO816.pdf 20 145 Construction Plan Map 5 Main Ash Pond – Starter Dike 145 COO817.pdf 21 146 Other Contractor Work Limits Main Ash Pond – Starter Dike 146 COO818.pdf 22 147 Notes and Details Sediment and Erosion Control Main Ash Pond – Starter Dike 147 COO822.pdf 23 148 Piezometer Details Main Ash Pond – Starter Dike 148 COO823.pdf 24 149 Surface Drainage Layout and Details Main Ash Pond – Starter Dike 149 C00826.pdf 25 150 Surface Drainage Layout and Details Main Ash Pond – Starter Dike *150 C00827.pdf* 26 151 West Side Sump Layout and Plan View Main Ash Pond – Starter Dike 151 COO828.pdf 27 152 Typical Cross Sections Main Ash Pond – Starter Dike 152 C00830.pdf 28 153 Typical Cross Sections Main Ash Pond – Starter Dike 153 C00831.pdf 29 154 Typical Cross Sections Main Ash Pond – Starter Dike 154 C00832.pdf 30 155 Typical Cross Sections Main Ash Pond – Starter Dike 155 C00833.pdf 31 156 Profile – Starter Dike - Main Ash Pond – Starter Dike *156 C00834.pdf* 32 157 Details Foundation Treatment Main Ash Pond – Starter Dike 157 COD835.pdf 33 158 Details Embankment Liner System Main Ash Pond – Starter Dike 158 COO836.pdf 34 159 Profile – Principal Spillway Main Ash Pond – Starter Dike *159 C00839.pdf* 35 160 Profile – Secondary Spillway Main Ash Pond – Starter Dike *160 C00840.pdf* 36 161 Details Riser Structure Main Ash Pond – Starter Dike 161 COO841.pdf 37 162 Details Riser Structure Main Ash Pond – Starter Dike 162 COO842.pdf 38 163 Details Riser Structure Main Ash Pond – Starter Dike *163 C00843.pdf* 39 164 Details Riser Structure Main Ash Pond – Starter Dike 164 COO844.pdf

### E. W. Brown Main Ash Pond

### Appendix A

### Table of Contents

Starter Dike  186 Embankment Cross Sections Sta. 503+00 Through Sta. 505+50 186 C00871.pdf Main Ash Pond – Starter Dike  187 Embankment Cross Sections Sta. 506+00 Through Sta. 508+50 187 C00872.pdf Main Ash Pond – Starter Dike  188 Embankment Cross Sections Sta. 509+00 Through Sta. 511+50 188 C00873.pdf Main Ash Pond – Starter Dike  189 Embankment Cross Sections Sta. 512+00 Through Sta. 514+50 189 C00874.pdf Main Ash Pond – Starter Dike  190 Embankment Cross Sections Sta. 515+00 Through Sta. 517+50 190 C00875.pdf Main Ash Pond – Starter Dike  191 Embankment Cross Sections Sta. 518+00 Through Sta. 520+50 191 C00876.pdf Main Ash Pond – Starter Dike  192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 192 C00877.pdf Main Ash Pond – Starter Dike  193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 193 C00878.pdf Main Ash Pond – Starter Dike  194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf Main Ash Pond – Starter Dike  195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf Main Ash Pond – Starter Dike	50 51 52 53 54 55 56 57 58 59 60	40 41 42 43 44 45 46 47 48 49	165 Details Riser Structure Main Ash Pond – Starter Dike 165 C00845.pdf 166 Access Bridge Profile Riser Structure Main Ash Pond – Starter Dike 166 C00846.pdf 167 Access Bridge Pier Details Main Ash Pond – Starter Dike 167 C00847.pdf 168 Hydraulic and Hydrologic Data Main Ash Pond – Starter Dike 168 C00851.pdf 169 Plan View Boring Layout – Map 1 Main Ash Pond – Starter Dike 169 C00852.pdf 170 Plan View Boring Layout – Map 2 Main Ash Pond – Starter Dike 170 C00853.pdf 171 Plan View Boring Layout – Map 3 Main Ash Pond – Starter Dike 170 C00854.pdf 172 Plan View Boring Layout – Map 4 Main Ash Pond – Starter Dike 172 C00855.pdf 173 Plan View Boring Layout – Map 5 Main Ash Pond – Starter Dike 173 C00856.pdf 174 Logs of Borings Main Ash Pond Perimeter (RD) Main Ash Pond – Starter Dike 174 C00857.pdf 175 Logs of Borings Main Pond Borrow Area (MP) Main Ash Pond – Starter Dike 176 C00858.pdf 176 Logs of Borings Main Pond Borrow Area (ASH) Main Ash Pond – Starter Dike 176 C00859.pdf 177 Logs of Borings Houp Property Borrow Area (BA) Main Ash Pond – Starter Dike 177 C00860.pdf 178 Logs of Borings Houp Property Borrow Area (BA) Main Ash Pond – Starter Dike 179 C00862.pdf 180 Logs of Borings Houp Property Borrow Area (BA) Main Ash Pond – Starter Dike 179 C00862.pdf 181 Logs of Borings Riser Structure (RS) Main Ash Pond – Starter Dike 180 C00863.pdf 182 Stability Analyses Section A – Sta. 511+78 Main Ash Pond – Starter Dike 182 C00865.pdf 183 Stability Analyses Section C – Sta. 580+70 Main Ash Pond – Starter Dike 184 C00867.pdf 185 Embankment Cross Sections Sta. 500+00 Through Sta. 502+50 185 C00870.pdf
62       187 Embankment Cross Sections Sta. 506+00 Through Sta. 508+50 187 C00872.pdf         63       188 Embankment Cross Sections Sta. 509+00 Through Sta. 511+50 188 C00873.pdf         64       189 Embankment Cross Sections Sta. 512+00 Through Sta. 514+50 189 C00874.pdf         65       190 Embankment Cross Sections Sta. 515+00 Through Sta. 517+50 190 C00875.pdf         66       191 Embankment Cross Sections Sta. 518+00 Through Sta. 520+50 191 C00876.pdf         67       192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 192 C00877.pdf         68       193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 193 C00878.pdf         69       194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf         69       194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf         69       195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf         70       195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf	61		186 Embankment Cross Sections Sta. 503+00 Through Sta. 505+50 <i>186 C00871.pdf</i>
Main Ash Pond – Starter Dike  189 Embankment Cross Sections Sta. 512+00 Through Sta. 514+50 189 C00874.pdf Main Ash Pond – Starter Dike  190 Embankment Cross Sections Sta. 515+00 Through Sta. 517+50 190 C00875.pdf Main Ash Pond – Starter Dike  191 Embankment Cross Sections Sta. 518+00 Through Sta. 520+50 191 C00876.pdf Main Ash Pond – Starter Dike  192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 192 C00877.pdf Main Ash Pond – Starter Dike  193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 193 C00878.pdf Main Ash Pond – Starter Dike  194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf Main Ash Pond – Starter Dike  195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf	62		
Main Ash Pond – Starter Dike  190 Embankment Cross Sections Sta. 515+00 Through Sta. 517+50 190 C00875.pdf Main Ash Pond – Starter Dike  191 Embankment Cross Sections Sta. 518+00 Through Sta. 520+50 191 C00876.pdf Main Ash Pond – Starter Dike  192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 192 C00877.pdf Main Ash Pond – Starter Dike  193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 193 C00878.pdf Main Ash Pond – Starter Dike  194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf Main Ash Pond – Starter Dike  195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf	63		
Main Ash Pond – Starter Dike  191 Embankment Cross Sections Sta. 518+00 Through Sta. 520+50 /9/ C00876.pdf Main Ash Pond – Starter Dike  67 192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 /92 C00877.pdf Main Ash Pond – Starter Dike  68 193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 /93 C00878.pdf Main Ash Pond – Starter Dike  69 194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 /94 C00879.pdf Main Ash Pond – Starter Dike  70 195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 /95 C00880.pdf	64		
<ul> <li>191 Embankment Cross Sections Sta. 518+00 Through Sta. 520+50 /9/ C00876.pdf Main Ash Pond – Starter Dike</li> <li>192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 /92 C00877.pdf Main Ash Pond – Starter Dike</li> <li>193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 /93 C00878.pdf Main Ash Pond – Starter Dike</li> <li>194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 /94 C00879.pdf Main Ash Pond – Starter Dike</li> <li>195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 /95 C00880.pdf</li> </ul>	65		190 Embankment Cross Sections Sta. 515+00 Through Sta. 517+50 <i>190 C00875.pdf</i>
<ul> <li>192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 192 C00877.pdf Main Ash Pond – Starter Dike</li> <li>193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 193 C00878.pdf Main Ash Pond – Starter Dike</li> <li>194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf Main Ash Pond – Starter Dike</li> <li>195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf</li> </ul>	66		191 Embankment Cross Sections Sta. 518+00 Through Sta. 520+50 <i>191 C00876.pdf</i>
<ul> <li>193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 193 C00878.pdf Main Ash Pond – Starter Dike</li> <li>194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf Main Ash Pond – Starter Dike</li> <li>195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf</li> </ul>	67		192 Embankment Cross Sections Sta. 521+00 Through Sta. 523+50 <i>192 C00877.pdf</i>
<ul> <li>194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 194 C00879.pdf</li> <li>Main Ash Pond – Starter Dike</li> <li>195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 195 C00880.pdf</li> </ul>	68		193 Embankment Cross Sections Sta. 524+00 Through Sta. 526+50 <i>193 C00878.pdf</i>
70 195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 <i>195 C00880.pdf</i>	69		194 Embankment Cross Sections Sta. 527+00 Through Sta. 529+50 <i>194 C00879.pdf</i>
	70		195 Embankment Cross Sections Sta. 530+00 Through Sta. 532+50 <i>195 C00880.pdf</i>

### E. W. Brown Main Ash Pond

### Appendix A

### Table of Contents

71	196 Embankment Cross Sections Sta. 533+00 Through Sta. 535+50 <i>196 C00881.pdf</i> Main Ash Pond – Starter Dike
72	197 Embankment Cross Sections Sta. 536+00 Through Sta. 541+50 <i>197 C00882.pdf</i> Main Ash Pond – Starter Dike
73	198 Embankment Cross Sections Sta. 539+00 Through Sta. 502+50 <i>198 C00882.pdf</i> Main Ash Pond – Starter Dike
74	199 Embankment Cross Sections Sta. 542+00 Through Sta. 544+50 <i>199 C00884.pdf</i> Main Ash Pond – Starter Dike
75	200 Embankment Cross Sections Sta. 545+00 Through Sta. 547+50 <i>200 C00885.pdf</i> Main Ash Pond – Starter Dike
76	201 Embankment Cross Sections Sta. 548+00 Through Sta. 550+50 <i>201 C00886.pdf</i> Main Ash Pond – Starter Dike
77	202 Embankment Cross Sections Sta. 551+00 Through Sta. 553+50 <i>202 C00887.pdf</i> Main Ash Pond – Starter Dike
78	203 Embankment Cross Sections Sta. 554+00 Through Sta. 556+50 <i>203 C00888.pdf</i> Main Ash Pond – Starter Dike
79	204 Embankment Cross Sections Sta. 557+00 Through Sta. 559 <i>204 C00889.pdf</i> Main Ash Pond – Starter Dike
80	205 Embankment Cross Sections Sta. 560+00 Through Sta. 562+50 <i>205 C00890.pdf</i> Main Ash Pond – Starter Dike
81	206 Embankment Cross Sections Sta. 563+00 Through Sta. 565+50 <i>206 C00891.pdf</i> Main Ash Pond – Starter Dike
82	207 Embankment Cross Sections Sta. 556+00 Through Sta. 568+50 <i>207 C00892.pdf</i> Main Ash Pond – Starter Dike
83	208 Embankment Cross Sections Sta. 569+00 Through Sta. 571+50 <i>208 C00893.pdf</i> Main Ash Pond – Starter Dike
84	209 Embankment Cross Sections Sta. 572+00 Through Sta. 574+50 <i>209 C00894.pdf</i> Main Ash Pond – Starter Dike
85	210 Embankment Cross Sections Sta. 575+00 Through Sta. 577+50 <i>210 C00895.pdf</i> Main Ash Pond – Starter Dike
86	211 Embankment Cross Sections Sta. 578+00 Through Sta. 580+50 <i>211 C00896.pdf</i> Main Ash Pond – Starter Dike
87	212 Embankment Cross Sections Sta. 581+00 Through Sta. 582+50 <i>212 C00897.pdf</i> Main Ash Pond – Starter Dike



Design Report

Main Ash Pond Expansion E.W. Brown Generating Station Kentucky Utilities Company Burgin, Mercer County, Kentucky

Prepared for: E.ON U.S. Louisville, Kentucky

October 19, 2007



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October 19, 2007

O.1.1.LX2007193R01

Mr. Jeffrey B. Heun, PE Project Manager E.ON U.S. Project Engineering Site Office E.W. Brown Generating Station 815 Dix Dam Road Harrodsburg, Kentucky 40330

Re:

Design Report

Main Ash Pond Expansion E.W. Brown Generating Station Kentucky Utilities Company Burgin, Mercer County, Kentucky

Dear Mr. Heun:

Fuller, Mossbarger, Scott an May Engineers, Inc. (FMSM) has prepared a Final Design Report for the proposed Main Ash Pond Starter Dike and Phased Expansion Project at the E.W. Brown Generating Station. This report presents an overview of the facility and the proposed impoundment, along with narrative discussions of relevant design issues and analysis methods.

If you have any questions or need more information, please feel free to call our office.

Respectfully submitted,

FULLER, MOSSBARGER, SCOTT AND MAY ENGINEERS. INC.

Vincent J. Severance, PE Senior Project Engineer

Dan A. Back, PE, PLS, SE

Project Manager

/vjs/ms

### **Design Report**

### Main Ash Pond Expansion E.W. Brown Generating Station Kentucky Utilities Company Burgin, Mercer County, Kentucky

### **Table of Contents**

Section	Page No.
1.	Introduction1
2.	Facility Description
	2.4. Main Ash Pond Phased Expansion
3.	Site Geology
4.	Main Ash Pond Phased Construction       6         4.1. General       6         4.2. Embankment Design       6         4.3. Liner Design       8         4.4. Outlet Works       8         4.5. Surface Drainage       8
5.	Engineering Analyses       9         5.1. Hydrologic Analyses       9         5.2. Stability Analyses       10         5.3. Liquefaction Analyses       11         5.4. Settlement Analyses       11    List of Tables
able	Page No.
able 1.	Anticipated Phased Embankment Crest Elevations6
able 1.	Summary of Watershed Data9
able 2.	Summary of 100 Year Storm Flood Routings
able 3.	Summary of PMP Flood Routings10
CINIO I.	Sammer J. St. 1 and T. Level 1 and Sammer Sa

### Table of Contents (Continued) List of Figures

Figure		Page No.
Figure 1.	Portion of the USGS Wilmore 7-1/2 Minute Topographic Quadrangle	2
Figure 2.	Portion of the USGS Wilmore 7-1/2 Minute Geologic Quadrangle	4
Figure 3.	Typical Section- Phased Expansion	7
	List of Appendixes	
Appendix		
Appendix A	Hydraulic Rating Curves Data and SITES Output	
Appendix B	Selection of Soil Strengths	
Appendix C	Selection of Seismic Design Parameters	
Appendix D	UTEXAS4 Output	
Appendix E	Liquefaction Analyses	
Appendix F	Settlement Calculations	
Appendix G	SITES Digital Data (CD)	

### Design Report

### Main Ash Pond Expansion E.W. Brown Generating Station Kentucky Utilities Company Burgin, Mercer County, Kentucky

### 1. Introduction

The E. W. Brown Generating Station, located in Mercer County, Kentucky, consists of three coal combustion steam turbine generating units. These units produce fly ash and bottom ash material which is currently stored in the existing Ash Treatment Basin (ATB). As the available storage volume is expended and a new flue gas desulphurization (FGD) facility, which will produce additional material, is constructed additional storage capacity will be required.

The initial steps in accommodating the enlargement of the storage facility have consisted of the development of an adjacent storage facility known as the Auxiliary Ash Pond and the design of a phased enlargement of the existing Main Ash Pond. Gypsum material from the FGD facility will be used to enlarge and raise the Main Ash Pond to provide additional fly ash and eventually gypsum material storage.

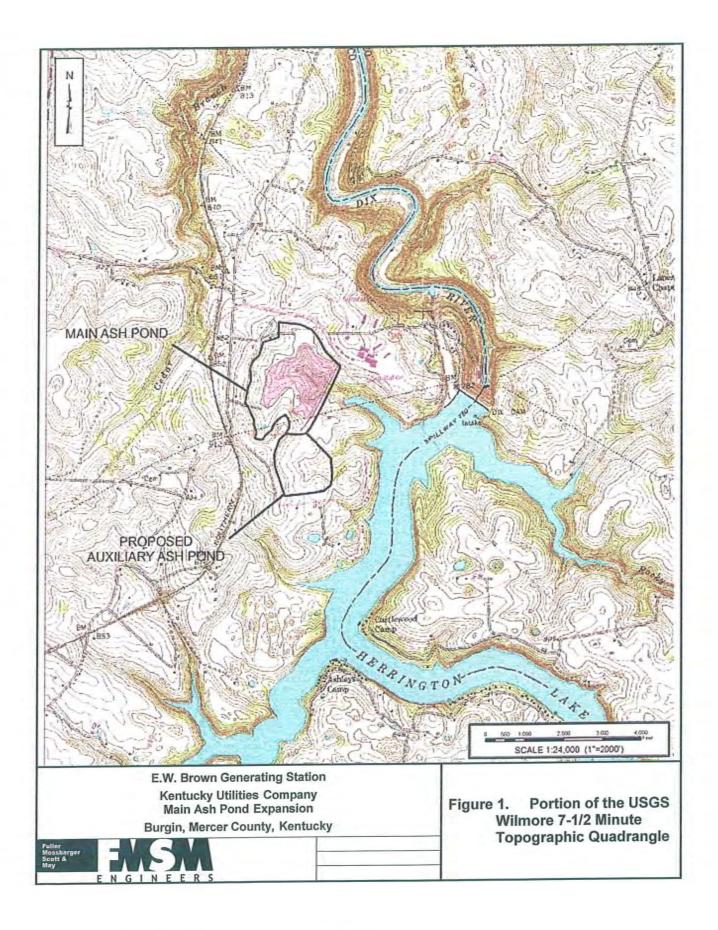
Each of the Main Ash Pond expansion phases is classified as a 'Class C – High Hazard' Structure. Design was accomplished in accordance with the Kentucky Division of Water (KYDOW) Engineering Memorandum No. 5 guidelines for a Class 'C' structure, Chapter 21, Section 4 of the "NRCS National Engineering Handbook" (NEH4) and the NRCS publication for "Design of Earth Dams and Reservoirs".

### 2. Facility Description

### 2.1. Existing Treatment Basin

The Ash Treatment Basin at the E. W. Brown Generating Station currently consists of a single pond which has been constructed with crest elevation rising at several times in the past. The present crest is at elevation 900 feet. The embankment consists primarily of a compacted clay embankment with additional zones of graded stone filters and shot rock drains. The present pond is not lined. A topographic map illustrating the site location is included as Figure 1.

Under current operating conditions the existing pond is projected to be totally full of settled fly ash and bottom ash some time in 2010. The gypsum material produced by the FGD facility, after it comes on line (currently planned in mid 2009) will increase the volume of material requiring storage and, if placed in the pond would shorten the life substantially.



### 2.2. Auxiliary Pond Construction

Work is currently underway to construct a second pond, known as the Auxiliary Pond. This facility is directly adjacent to the existing pond. The Auxiliary Pond will contain fly ash discharges for the time period that the existing pond is taken off-line for enlargement construction. It will also provide storage for the bottom ash produced until the year 2030. The Auxiliary Pond is scheduled to become operational at the end of January, 2008. For clarity, the existing pond is referred to as the "Main Ash Pond" throughout this report.

### 2.3. Initial Main Ash Pond Enlargement (Starter Dike)

When the Auxiliary Pond is placed in service and the plant discharges are directed into it, the Main Ash Pond will be dewatered and the initial phase of its expansion will be constructed. This phase of construction is known as the Main Ash Pond "Starter Dike" phase. It is also referred to as Phase 1. This construction will consist of regrading the ash deposits within the pond, construction of a new riser and outlet works, construction of an initial raised embankment to elevation 902 feet and installation of a geosynthetic flexible membrane liner (FML) system. The plans for this phase of construction are complete and the construction is scheduled to occur between February 2008 and December 2010.

### 2.4. Main Ash Pond Phased Expansion

The purpose of the phased expansion projects is to expand the Main Ash Pond Starter Dike in four distinct vertical phases (Phases 2, 3, 4 and 5) while the Main Ash Pond and Auxiliary Ash Pond remains fully operational. Phased expansion consists of expanding the gypsum embankment in four phases from elevation 902 to a final crest elevation 962 to provide additional storage for sluiced fly ash and gypsum until the year 2030. Each phase will include downstream gypsum embankment overlay construction and an extension of the compacted clay and FML liner system along the expanded upstream embankment slope.

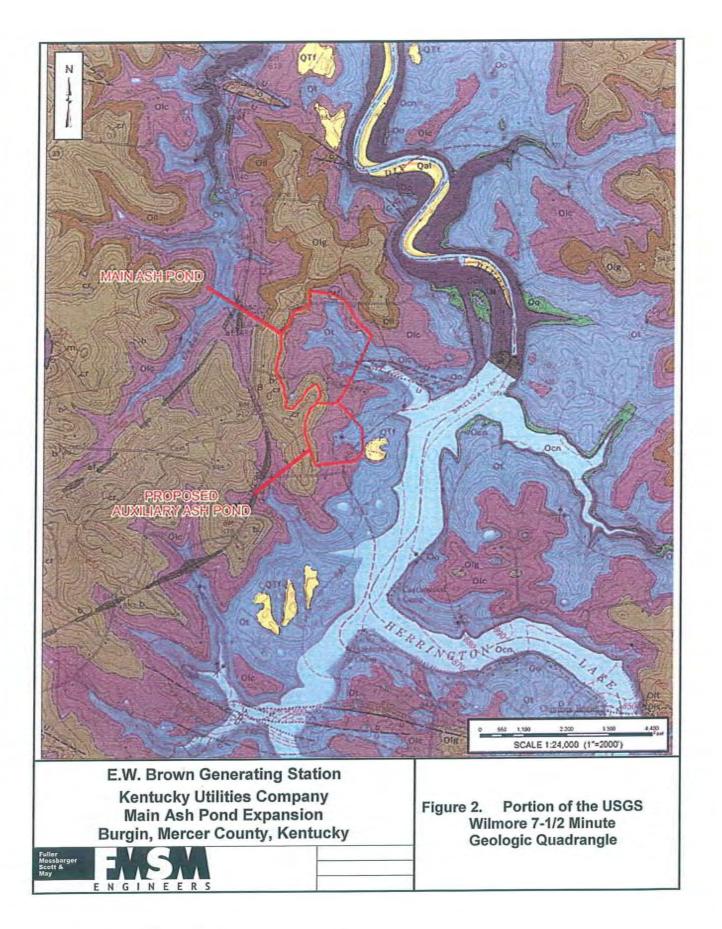
### 3. Site Geology

### 3.1. Geologic Setting

Available U.S. Geological Survey (USGS) geologic mapping (Geologic Map of the Wilmore, Kentucky Quadrangle, USGS, 1970) shows the site to be underlain by bedrock belonging to the Lexington Limestone and Tyrone Limestone Formations, both of which are Ordovician in age. Figure 2 shows the approximate location of the site on a portion of the Wilmore, Kentucky, USGS 7½-Minute Geologic Quadrangle. In the site area, the members of the Lexington Limestone present include (from top to bottom) the Grier Limestone Member (Labeled Olg on the mapping), Logana Member (Oll) and Curdsville Limestone Member (Olc). The Tyrone Limestone (Ot) underlies the Curdsville Limestone Member.

The Grier Limestone Member consists of light-gray, medium to coarse grained limestone that is bioclastic, poorly sorted and fossiliferous. Bedding thickness is typically less than one foot and beds of nodular, fossiliferous limestone with a micrograined calcite matrix are dispersed throughout.

The Logana Member consists of interbedded limestone and shale in three distinct zones, although this member grades into the Grier Member in portions of the quadrangle. The upper zone consists of two to three feet of light-olive-gray micrograined limestone interbedded with



olive-gray to medium-gray shale. The middle zone of five to eight feet of brachiopod coquina weathers to a pink-gray color. The lower zone is five to seven feet to interbedded limestone and shale that is similar to the upper zone.

The Curdsville Limestone Member consists of medium to light-gray, medium to coarse grained, bioclastic limestone. The lower part is well sorted and locally cross bedded and grades upward into an irregularly bedded, poorly sorted fossiliferous limestone. Chert nodules and silicified fossils are common in the lower part.

The Tyrone Limestone consists of two types of limestone. The first is light-gray to light-olive-gray cryptograined limestone that contains small areas of clear calcite or "birdseye" limestone. The second is light-gray to light-brownish-gray cryptograined limestone that contains small areas of micrograined, calcareous dolomite. In some portions of the quadrangle, up to three bentonite beds (each up to two feet thick) are found within the Tyrone. These bentonite layers are often underlain by thin chert layers. Persistent layers of argillaceous limestone and shale are present in the uppermost ten feet and middle of the unit.

Structure contours drawn on top of the Tyrone Limestone indicate a general rock strata dip of approximately 40 feet per mile towards northwest direction. The geologic mapping shows a graben-type feature immediately downstream of the ash treatment basin embankment, with a general northwest-southeast trend roughly aligned with the tributary. Several small normal faults are identified within the graben, resulting in the top of the Tyrone being as much as 50 feet lower within the graben than outside the graben. The mapped faults do not extend into the areas being considered for the proposed ash treatment basin extension.

### 3.2. Geotechnical Exploration

A field performance test was conducted during November 2005 within the existing ash pond to characterize engineering properties of the ash deposits and to evaluate the constructability of access roads and embankments on the dewatered ash surface. The testing provided valuable information regarding dewatering methods, pore pressure dissipation, haul road performance, dust control, and selection of geotechnical engineering properties of the ash deposits for final design. Refer to FMSM's report entitled "Geotechnical Characterization of Ash Basin Deposits" prepared on September 18, 2006 for detailed information.

Fourteen cone penetration tests (CPT) were completed within the Main Ash Pond. Eleven of the tests were advanced from various barge locations floating on the pond surface. The remaining three were pushed from access roads on the dewatered ash surface during the field performance test. In general the CPT results indicate the upper 10 to 50 ft of material appears to be mostly fly ash and the deeper penetrations indicated significantly more coarse material (bottom ash) in the lower elevations of the basin. This correlates well with the historical development of the basin which was originally constructed for bottom ash and was also used for fly ash at a later date.

FMSM also performed ten (10) rock core borings within the perimeter area of the Main Ash Pond to explore subsurface conditions and evaluate the bedrock. Based on the rock core borings, the top of the bedrock surface within the footprint of the Main Ash Pond

embankment expansion is relatively shallow and uniform. Depths to bedrock varied from 1.0 feet to 2.3 feet. Rock core samples indicate the underlying bedrock to be predominantly limestone; light gray in color, fine to coarse crystalline grained, hard, thin-bedded with shale stringers and partings.

Boring locations, logs of borings and other pertinent geotechnical data are presented in the construction plans.

### 4. Main Ash Pond Phased Construction

### 4.1. General

It is envisioned that the Main Ash Pond phased expansion will be constructed in five distinct vertical phases from elevation 900 feet to elevation 962 feet. The following minimum embankment crest elevations are anticipated for each phase of the Main Ash Pond expansion.

Table 1. Anticipated Phased Embankment Crest Elevations

Construction Phase	Minimum Crest Elevation (feet)
Phase 1 (Starter Dike)	902
Phase 2	912
Phase 3	928
Phase 4	946
Phase 5	962

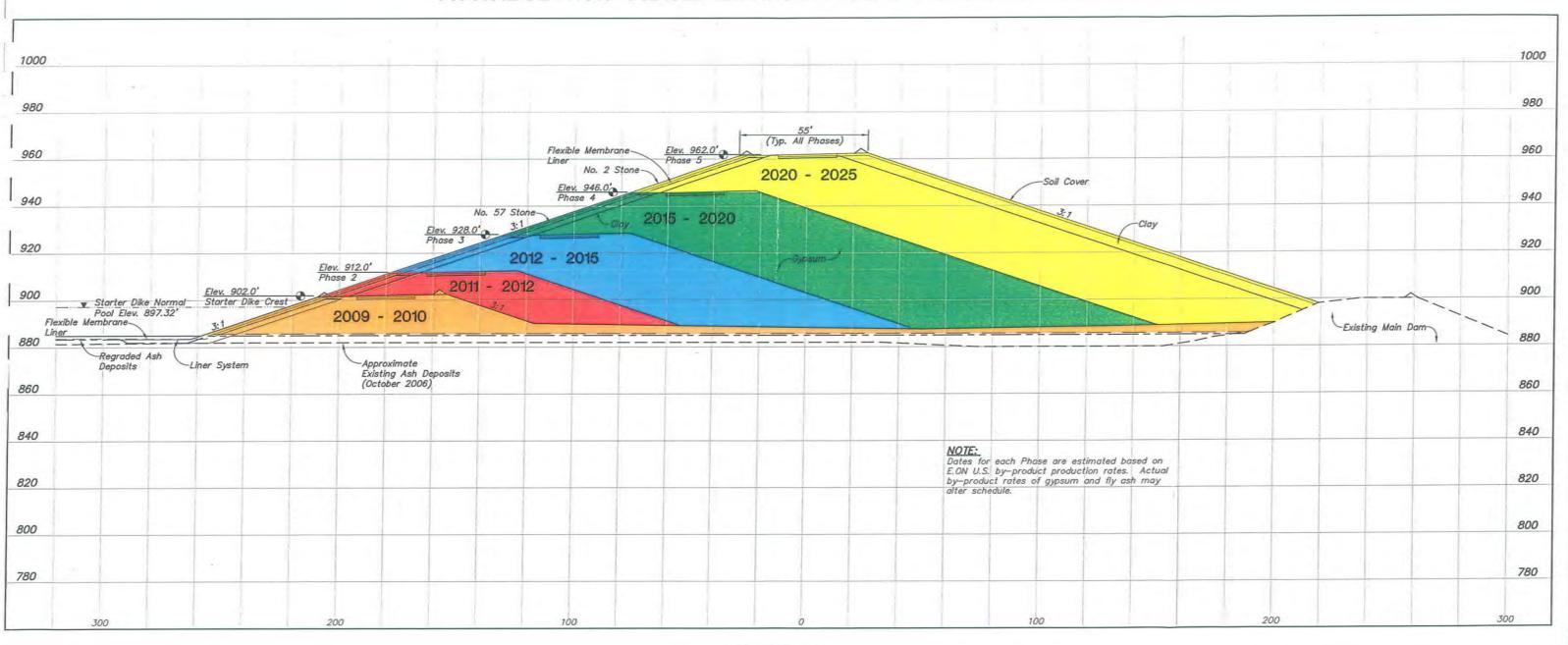
Each embankment phase will have a minimum crest width of 55 feet with 3H:1V embankment side slopes. A typical cross section of the phased expansion project is provided in Figure 3.

The Main Ash Pond will be closed in 2030 with a maximum sluiced fly ash/gypsum surface at approximate elevation of 953 feet allowing for five feet of free water and four feet of storm run-up and freeboard below the final crest elevation of 962 feet.

### 4.2. Embankment Design

Working elements of each phase will include downstream gypsum embankment overlay construction and extension of the compacted clay layer and FML along the upstream embankment slopes. It is anticipated that the structural lifts of gypsum will be placed in the applicable phased embankment expansion as gypsum is produced from the gypsum dewatering facility at a rate of approximately 1,000 to 1,500 cubic yards per day. In order to achieve the necessary minimum embankment crest heights during the phased construction, approximately 70 percent gypsum utilization is required for embankment construction. The remaining 30 percent of gypsum will be sluiced to the Main Ash Pond. This sluicing will primarily occur during winter months and periods of inclement weather when proper embankment placement is more difficult.

### TYPICAL SECTION - PHASED EXPANSION EMBANKMENT CONSTRUCTION



### 4.3. Liner Design

The expanded Main Ash Pond will be lined with a flexible membrane liner. The liner system will consist of 60 mil, textured, linear low density polyethylene (LLDPE) FML and two protective layers of thick filter fabric placed above and below the FML. During the initial Starter Dike phase construction, the existing ash pond and will be dewatered and the ash deposits will be stabilized and regraded. The liner system will be installed directly on the regraded ash deposits along the pond bottom, and upon a compacted clay zone constructed along the upstream face of the gypsum embankment. The clay layer will extend below the FML approximately 25 feet from the embankment toe into the regrade bottom pond. The FML and filter fabric system will be covered with eighteen inches of bottom ash along the pond bottom and by a 12-inch thick layer of No. 57 crushed stone and a 12-inch layer of No. 2 stone on the 3H:1V upstream slopes to protect the liner system and provide wave wash resistance.

### 4.4. Outlet Works

The pool elevation will be controlled by a new 72 feet tall riser structure using stop logs, sluice gates and spillway pipes to convey flow through the embankment. The new riser structure will be equipped with two principal spillway outlet pipes: (1) a 30-inch diameter outlet pipe to the existing KPDES discharge point and (2) a 30-inch diameter outlet pipe to the Auxiliary Ash Pond. The riser structure will be constructed of reinforced concrete with an 8 foot by 8 foot internal chamber. Stop logs will be inserted on one side of the riser structure to raise the pool as needed to maintain free water over the hydraulically placed ash deposits. Access to the riser will be provided via a bridge and a temporary scaffold stair tower. The existing riser structure will be partially removed and the existing outlet pipe will be grouted and abandoned.

The Main Ash Pond expansion will be constructed without an emergency spillway. FMSM and E.ON U.S. met with the KYDOW on March 8, 2007 to discuss the overall intent of the facility as a process pond and to request a waiver to eliminate the emergency spillway. KYDOW indicated that a waiver would be issued for the emergency spillway requirement provided the PMP design storm event can be safely routed through the new principal spillway riser structure during each phase of Main Ash Pond expansion project.

The 30-inch diameter outlet pipes will be gated with independently operated sluice gates. An un-gated 18-inch diameter overflow spillway pipe will be provided within the riser structure to convey flow directly to the 30-inch diameter Principal Spillway outlet pipe beyond the sluice gates in the event of sluice gate failures or if the gates are accidentally left closed. The overflow spillway pipe was sized to prevent a continuous process baseflow of 15.4 cfs from overtopping any of the embankment phases.

### 4.5. Surface Drainage

The impounded pool area serves as the primary watershed with limited perimeter areas that direct runoff into the pond during Phases 1 and 2. Beginning in Phase 3, the Main Ash Pond drainage area will consist solely of the area contained within the crest limits of the embankment. Storm water runoff from the downstream face of gypsum embankments will be collected by a series of ditches, collection ponds, pipes and wet wells and pumped primarily to the Auxiliary Ash Pond or possibly to the Main Ash Pond. Final downstream embankment slopes will be covered with a four foot layer of compacted clay and a one-foot layer of topsoil.

### 5. Engineering Analyses

### 5.1. Hydrologic Analyses

Hydrologic and hydraulic design consisted of flood routing techniques and peak flow evaluations for each embankment construction phase. The U.S. Department of Agriculture SITES computer application was used to analyze each embankment phase. A digital version of the SITES data files is included on a CD in Appendix G of this report.

The Kentucky Division of Water (KYDOW) Engineering Memorandum No. 5 guidelines for a Class 'C' structure were used to select design storms for the analyses. The design guidelines also conform with Chapter 21, Section 4 of the "NRCS National Engineering Handbook" (NEH4) and the NRCS TR-60 publication for "Design of Earth Dams and Reservoirs".

Principal Spillway stage-discharge rating curves were calculated for each embankment phase to route the Principal Spillway Hydrograph (PSH) and Freeboard Hydrograph (FBH) events through the Main Ash Pond. A summary of the watershed data for each embankment phase is presented in Table 1. The Principal Spillway riser structure was designed using overflow weir equations and pipe culvert discharge rating curves for the outlet pipes developed within an Excel spreadsheet. The riser rating curve calculations for each embankment phase are included in Appendix A.

The results of PSH and FBH routings are presented in Table 2 and Table 3 and on sheets BR0-C-00851, BR0-C-00984 and BR0-C-00985 of the plans, and SITES output files are included in Appendix A. All flood routings were started with a pre-existing baseflow of 15.45 cfs passing over the riser stoplogs and through the 30-inch discharge pipe to the Auxiliary Ash Pond. No emergency spillway hydrographs were developed because the understanding that the KYDOW will waive the requirement for an emergency spillway for the process pond provided the PMP design storm event can be safely routed through the Main Ash Pond.

Table 2. Summary of Watershed Data

Watershed Data	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5
Drainage Area (acres)	102.1	102.6	94.7	110.9	118.7
Time of Concentration (hours)	0.25	0.25	0.25	0.25	0.25
Runoff Curve Number	95	95	95	95	95
Top of Dam Elevation (feet)	902	912.0	928.0	946.0	962.0
Top Stoplog Elevation (feet)	896.0	906.5	923.0	941.0	957.0
Normal Pool Elev. with Baseflow (feet)	897.55	907.90	924.40	942.40	958.16
Normal Pool Impounded Area (acres)	73.45	80.14	88.50	97.87	106.42
Normal Pool Impounded Volume (ac-ft)	868	1,655	3,062	4,740	6,350
Baseflow (cfs)	15.45	15.45	15.45	15.45	15.45

Table 3. Summary of 100 Year Storm Flood Routings

Flood routing Data	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5
Precipitation (inches)	4.53	4.53	4.53	4.53	4.53
Runoff (inches)	3.95	3.95	3.95	3.95	3.95
Peak Inflow (cfs)	345	346	321	373	398
Peak Principal Spillway Outflow (cfs)	20.7	22.6	21.4	21.8	23.0
Maximum Water Surface Elevation (feet)	897.99	908.30	924.73	942.76	958.51
Freeboard (feet)	4.01	3.70	3.27	3.24	3.49

Table 4. Summary of PMP Flood Routings

Flood routing Data	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5
Precipitation (inches)	28.0	28.0	28.0	28.0	28.0
Runoff (inches)	27.4	27.4	27.4	27.4	27.4
Peak Inflow (cfs)	2159	2169	2003	2343	2507
Peak Principal Spillway Outflow (cfs)	44.7	76.8	65.4	69.4	81.6
Maximum Water Surface Elevation (feet)	900.56	910.58	926.67	944.82	960.52
Freeboard (feet)	1.44	1.42	1.33	1.18	1.48

### 5.2. Stability Analyses

Stability analyses were performed on the Phase 1 through 5 embankment configurations using the *UTEXAS4* computer program. The analyses were conducted to verify the long term stability of each embankment phase. Soil shear strength parameters used in the stability analyses were selected based on the different types of material to be incorporated into the embankment, the results of consolidated-undrained triaxial tests with pore pressure measurements or direct shear tests, and experience with coal combustion by-product materials on similar projects. The shear strength parameters used in the analyses are presented in Appendix B and on drawings BR0-C-00999 and BR0-C-01001.

Three (3) types of analyses were performed to evaluate the slope stability which included,

- Static Slope Stability Analysis
- Pseudo-Static Slope Stability Analysis without Liquefaction and
- Pseudo-Static Slope Stability Analysis with Liquefaction.

A summary of the selected seismic design parameters are presented in Appendix C.

To support the current design effort, three consolidated undrained triaxial compression tests were performed on compacted samples of gypsum (flue gas desulphurization product) provided by E.ON U.S. The test results indicated an effective stress strength envelope having  $\phi'$  of about 43°. However, given the friable nature of gypsum and the lack of field experience with large structures built of this material, a more conservative estimate of strength (c'=0,  $\phi'$ =35°) was used in the analysis. All existing ash basin deposits were

assumed to have the same long-term drained strength parameters as those determined during the 1989 design effort (c'=0,  $\phi$ '=32°). A summary of the gypsum laboratory testing results and the soil strengths used for slope stability analyses is included in Appendix B.

According to the U.S. Department of Agriculture, NRCS Technical Release No. 60, typical minimum factor of safety for earth dams and reservoirs are as follow:

- Long Term, upstream and downstream embankment slopes: Min. FS=1.4
- Steady Seepage with seismic forces (downstream slopes): Min. FS=1.1

The results from the *UTEXAS4* slope stability analysis and selected *UTEXAS4* output files have been provided in Appendix D. The minimum static factor of safety obtained in this study was 2.0 which occurred along the Phase 5 upstream face of Section 511+78. The minimum pseudo-static factor of safety obtained for this study was 1.3 which occurred along the Phase 5 downstream face of Section 527+80 during a pseudo-static analysis without liquefaction. The results of the stability analyses are presented graphically on drawings BR0-C-00999 through BR0-C-01001.

### 5.3. Liquefaction Analyses

Liquefaction analyses were performed on a critical section at Station 527+80 where saturated ash deposits extend beneath the full width of the existing north embankment. A summary of the methods used to access the potential for liquefaction beneath the expanded Main Ash Pond and the results of this evaluation are presented in Appendix E. The analyses indicated a potential for liquefaction of the saturated ash deposits when the phreatic surface is above elevation 856.0 feet. Liquefaction in this area could destabilize the existing dike and could cause progressive sliding of the larger embankment.

Groundwater levels within the ash deposits are expected to gradually recede after the Main Ash Pond is taken out of service, and groundwater levels should continue to recede during construction and operation of the expansion projects after the liner system is constructed. The construction drawings include a plan to monitor groundwater levels beneath the dike along the north end of the basin during Phases 1 and 2. If the measured ground water table does not drop to an elevation of 856 feet or lower, a French drain system will be installed during the Phase 2 construction project to lower the water level and stabilize the embankment against a potential liquefaction failure.

### 5.4. Settlement Analyses

A settlement analysis was conducted to determine the amount of long term settlement expected during the phased embankment construction. Two settlement profiles were analyzed across some of the deepest existing ash deposits. Compressibility parameters measured for the ash deposits during the field performance test were used in the computations. (Initial Void Ratio, e<sub>0</sub>=1.14; Compression Index, C<sub>c</sub>=0.28). The settlement analysis calculations have been provided in Appendix F.

Consolidation of the existing ash deposits was considered to be the main component of postconstruction settlement. Settlement computations indicate that the existing ash deposits will settle up to as much as eight (8) feet under the loading of the constructed embankment phases and the sluiced waste material. The liner system will experience differential vertical settlements of over eight feet across each settlement profile. The actual liner deformation will occur in a bowl shaped pattern across the existing 110 acre ash pond. Values of liner strain due to differential settlement are expected to be less that 0.5 percent which are well below typical values of allowable elongation and biaxial strain for LLDPE FML materials.

The rate of settlement will depend upon the rate of gypsum embankment construction and the rate at which the groundwater level recedes after the liner system is installed. While this settlement will have some affect on actual material quantities, it is believed that the continuous nature of the embankment placement will allow the planned crest to be maintained without difficulty.

Appendix A

Hydraulic Rating Curves Data and SITES Output

Contour (Elev.)	Area (Acres)	Area (SF)	Volume (Cu. yd.)	Cum. Vol. (Cu. yd.)	Volume (Ac-ft)	Cum. Vol
885	64.477	2,808,615	0	0	0.0	0.0
886	65.383	2,848,072	104,753	104,753	64.9	64.9
888	66.956	2,916,610	213,507	318,260	132.3	197.3
890	68.095	2,966,229	217,883	536,143	135.1	332.3
892	69.471	3,026,156	221,940	758,083	137.6	469.9
894	70.905	3,088,616	226,473	984,556	140.4	610.3
896	72.356	3,151,846	231,128	1,215,685	143.3	753.5
898	73.765	3,213,221	235,743	1,451,428	146.1	899.6
900	75.121	3,272,252	240,203	1,691,631	148.9	1048.5
902	76.422	3,328,959	244,489	1,936,120	151.5	1200.1
904	77.739	3,386,329	248,714	2,184,834	154.2	1354.2
906	79.015	3,441,884	252,897	2,437,731	156.8	1511.0
908	80.167	3,492,094	256,814	2,694,545	159.2	1670.2
910	81.298	3,541,328	260,497	2,955,042	161.5	1831.6
912	82.290	3,584,538	263,921	3,218,963	163.6	1995.2
914	83.282	3,627,748	267,122	3,486,085	165.6	2160.8
916	84.274	3,670,958	270,322	3,756,407	167.6	2328.4
918	85.266	3,714,168	273,523	4,029,930	169.5	2497.9
920	86.258	3,757,378	276,724	4,306,654	171.5	2669.4
922	87.276	3,801,757	279,968	4,586,622	173.5	2842.9
924	88.295	3,846,136	283,255	4,869,878	175.6	3018.5
926	89.314	3,890,514	286,543	5,156,420	177.6	3196.1
928	90.333	3,934,893	289,830	5,446,250	179.6	3375.8
930	91.352	3,979,272	293,117	5,739,367	181.7	3557.5
932	92.397	4,024,821	296,448	6,035,815	183.7	3741.2
934	93.443	4,070,370	299,822	6,335,637	185.8	3927.0
936	94.488	4,115,919	303,196	6,638,833	187.9	4115.0
938	95.534	4,161,468	306,570	6,945,403	190.0	4305.0
940	96.580	4,207,017	309,944	7,255,347	192.1	4497.1
942	97.652	4,253,716	313,360	7,568,707	194.2	4691.3
944	98.724	4,300,416	316,820	7,885,527	196.4	4887.7
946	99.796	4,347,115	320,279	8,205,806	198.5	5086.2
948	100.868	4,393,815	323,738	8,529,544	200.7	5286.9
950	101.940	4,440,514	327,197	8,856,741	202.8	5489.7
952	103.038	4,488,337	330,698	9,187,439	205.0	5694.7
954	104.136	4,536,159	334,241	9,521,680	207.2	5901.9
956	105.234	4,583,982	337,783	9,859,463	209.4	6111.2
958	106.332	4,631,804	341,325	10,200,788	211.6	6322.8
960	107.429	4,679,627	344,868	10,545,656	213.8	6536.6
961.41	108.216	4,713,878	245,275	10,790,931	152.0	6688.6

## PIPE CULVERT ANALYSIS COMPUTATION OF CULVERT PERFORMANCE CURVE

### October 4, 2007

PROGRAM INPUT DATA	VALUE
DESCRIPTION	VALUE
Culvert Diameter (ft)  FHWA Chart Number.  FHWA Scale Number (Type of Culvert Entrance)  Manning's Roughness Coefficient (n-value).  Entrance Loss Coefficient of Culvert Opening  Culvert Length (ft)  Invert Elevation at Downstream end of Culvert (ft)  Invert Elevation at Upstream end of Culvert (ft)  Culvert Slope (ft/ft)	2.45 2 1 0.012 0.1 115.0 891.79 895.0 0.0279
Starting Flow Rate (cfs)  Incremental Flow Rate (cfs)  Ending Flow Rate (cfs)  Starting Tailwater Depth (ft)  Incremental Tailwater Depth (ft)  Ending Tailwater Depth (ft)	1.0 1.0 61.0 0.0 0.0

### COMPUTATION RESULTS

Flow Rate (cfs)	Tailwater Depth (ft)	Headwater Inlet Control	(ft). Outlet Control	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Velocity (fps)	
 1.0	0.0	0.4	0.0	0.2	0.33	0.2	5.35	
2.0	0.0	0.59	0.0	0.28	0.46	0.28	6.55	
3.0	0.0	0.74	0.0	0.35	0.57	0.35	7.41	
4.0	0.0	0.87	0.0	0.4	0.66	0.4	8.08	
5.0	0.0	0.98	0.0	0.44	0.74	0.44	8.62	
6.0	0.0	1.09	0.0	0.48	0.82	0.48	9.09	
7.0	0.0	1.19	0.0	0.52	0.88	0.52	9.52	
8.0	0.0	1.29	0.0	0.56	0.95	0.56	9.9	
9.0	0.0	1.38	0.0	0.59	1.01	0.59	10.25	
10.0	0.0	1.47	0.0	0.62	1.06	0.62	10.56	
11.0	0.0	1.55	0.0	0.65	1.12	0.65	10.87	
12.0	0.0	1.64	0.0	0.69	1.17	0.69	11.13	
13.0	0.0	1.72	0.0	0.71	1.22	0.71	11.4	
14.0	0.0	1.8	0.0	0.74	1.27	0.74	11.63	
15.0	0.0	1.89	0.0	0.77	1.31	0.77	11.86	
16.0	0.0	1.97	0.0	0.8	1.36	0.8	12.07	
17.0	0.0	2.05	0.0	0.82	1.4	0.82	12.3	
18.0	0.0	2.13	0.0	0.85	1.45	0.85	12.48	
19.0	0.0	2.21	0.0	0.87	1,49	0.87	12.66	
20.0	0.0	2.28	0.0	0.89	1.53	0.89	12.86	
21.0	0.0	2.36	0.0	0.92	1.57	0.92	13.03	
22.0	0.0	2.44	0.0	0.94	1.6	0.94	13.19	
23.0	0.0	2.52	0.0	0.96	1.64	0.96	13.34	
24.0	0.0	2.6	0.0	0.99	1.68	0.99	13.5	
25.0	0.0	2.68	0.0	1.01	1.71	1.01	13.65	
26.0	0.0	2.78	0.0	1.03	1.75	1.03	13.81	
27.0	0.0	2.92	0.0	1.05	1.78	1.05	13.94	
28.0	0.0	3.04	0.0	1.07	1.81	1.07	14.08	
29.0	0.0	3.11	0.0	1.1	1.84	1.1	14.21	
30.0	0.0	3.19	0.0	1.12	1.88	1.12	14.33	
31.0	0.0	3.3	0.0	1.14	1.91	1.14		
32.0	0.0	3.4	0.0	1.16	1.93	1.16	14.56	
33.0	0.0	3.51	0.0	1.18	1.96	1.18	14.69	
34.0	0.0	3.63	0.0	1.2	1.99	1.2	14.79	
35.0	0.0	3.75	0.0	1.22	2.01	1.22	14.9	
36.0	0.0	3.87	0.0	1.24	2.04	1.24	15.01	

37.0	0.0	3.99	0.0	1.26	2.06	1.26	15.1
38.0	0.0	4.12	0.0	1.28	2.09	1.28	15.2
39.0	0.0	4.25	0.0	1.3	2.11	1.3	15.3
40.0	0.0	4.38	0.0	1.32	2.13	1.32	15.39
	0.0	4.52	0.0	1.34	2.15	1.34	15.49
41.0 42.0	0.0	4.66	0.0	1.36	2.17	1.36	15.57
	0.0	4.81	0.0	1.38	2.18	1.38	15.66
43.0	0.0	4.96	0.0	1.4	2.2	1.4	15.74
	0.0	5.11	0.0	1.42	2.22	1.42	15.82
45.0	0.0	5.26	0.0	1.44	2.23	1.44	15.9
46.0	0.0	5.42	0.0	1.46	2.25	1.46	15.98
47.0	0.0	5.59	0.0	1.49	2.26	1.49	16.06
48.0	0.0	5.75	0.0	1.51	2.27	1.51	16.13
49.0		5.92	0.0	1.53	2.28	1.53	16.2
50.0	0.0	6.09	0.0	1.55	2.29	1.55	16.26
51.0	0.0	6.27	0.0	1.57	2.3	1.57	16.33
52.0	0.0	6.45	0.0	1.59	2.31	1.59	16.39
53.0	0.0	6.63	0.0	1.61	2.32	1.61	16.44
54.0	0.0	6.81	0.0	1.63	2.33	1.63	16.51
55.0	0.0	7.0	0.0	1.65	2.34	1.65	16.57
56.0	0.0	7.2	0.0	1.67	2.35	1.67	16.61
57.0	0.0		0.0	1.69	2.35	1.69	16.67
58.0	0.0	7.39	0.0	1.72	2.36	1.72	16.71
59.0	0.0	7.59		1.74	2.36	1.74	16.76
60.0	0.0	7.8	0.0	1.76	2.37	1.76	16.8
61.0	0.0	8.0	0.0	1.70	2.31	2.10	

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## PIPE CULVERT ANALYSIS COMPUTATION OF CULVERT PERFORMANCE CURVE

### October 4, 2007

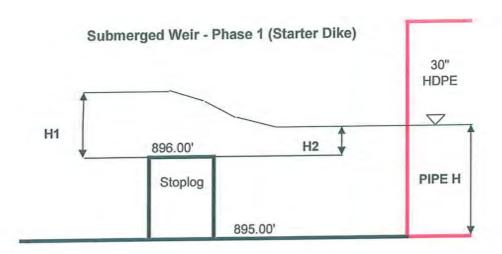
PROGRAM INPUT DATA	
DESCRIPTION	VALUE
Culvert Diameter (ft)	2.45 2 1 0.012 0.1 115.0 891.79 895.0 0.0279
Starting Flow Rate (cfs)	60.0 1.0 120.0
Starting Tailwater Depth (ft) Incremental Tailwater Depth (ft) Ending Tailwater Depth (ft)	0.0 0.0 0.0

### COMPUTATION RESULTS

		C	OMPUTATION				0.67+4
Flow T	ailwater	Headwate			Critical		Velocity
Rate	Depth	Inlet	Outlet				(fps)
(cfs)	(ft)	Control	Control	(ft)	(ft)	(10)	(100)
 60.0	0.0	7.8	0.0	1.74		1.74	16.76
61.0	0.0	8.0	0.0	1.76	2.37	1.76	16.8
62.0	0.0	8.21	0.0	1.79	2.37		16.84
63.0	0.0	8.42	0.0	1.81	2.38		16.88
64.0	0.0	8.64	0.0	1.83	2.38		16.91
65.0	0.0	8.86	0.0	1.86	2.39		16.94
66.0	0.0	9.08	0.0	1.88			16.96
67.0	0.0	9.31	0.0	1.91	2.39		16.99
68.0	0.0	9.54	0.0	1.94	2.4	1.94	17.0
69.0	0.0	9.78		1.97	2.4	1.97	17.0
70.0	0.0	10.01	0.0	2.0	2.4	2.0	17.01
	0.0	10.25		2.03	2.4	2.03	17.0
71.0 72.0	0.0	10.5	0.0	2.06	2.41	2.06	16.99
73.0	0.0	10.74	0.0	2.1	2.41	2.1	16.95
74.0	0.0	10.99	0.0	2.15	2.41	2.15	16.89
	0.0	11.25	0.0	2.21	2.41	2.21	16.77
75.0 76.0	0.0	11.51	3.38	2.45	2.42	2.45	16.12
77.0	0.0	11.77	3.49	2.45	2.42	2.45	16.33
78.0	0.0	12.03	3.6	2.45	2.42	2.45	16.55
	0.0	12.3	3.72	2.45	2.42	2.45	16.76
79.0	0.0	12.57	3.83	2.45	2.42	2.45	16.97
80.0	0.0	12.84	3.95	2.45	2.42	2.45	17.18
82.0	0.0	13.12	4.07	2.45	2.42	2.45	17.39
	0.0	13.4	4.18	2.45	2.43	2.45	17.61
83.0	0.0	13.69	4.3	2.45	2.43	2.45	17.82
84.0 85.0	0.0	13.98		2.45	2.43	2.45	18.03
	0.0	14.27	4.55	2.45	2.43	2.45	18.24
86.0	0.0	14.56	4.67	2.45	2.43	2.45	18.45
87.0	0.0	14.86	4.8	2.45	2.43	2.45	18.67
88.0	0.0	15.16	4.93	2.45	2.43	2.45	18.88
89.0	0.0	15.47	5.06	2.45	2.43	2.45	19.09
90.0	0.0		5.19	2.45	2.43	2.45	19.3
91.0	0.0	16.09	5.32	2.45		2.45	19.51
 92.0	0.0	16.41	5.45	2.45		2.45	19.73
93.0	0.0	16.72	5.59	2.45	2.44	2.45	19.94
94.0	0.0	17.05	5.72	2.45	2.44		20.15
95.0	0.0	17.05	3.14	2.25			

96.0	0.0	17.37	5.86	2.45	2.44	2.45	20.36
97.0	0.0	17.7	6.0	2.45	2.44	2.45	20.58
98.0	0.0	18.03	6.14	2.45	2.44	2.45	20.79
99.0	0.0	18.37	6.28	2.45	2.44	2.45	21.0
100.0	0.0	18.71	6.42	2.45	2.44	2.45	21.21
101.0	0.0	19.05	6.57	2.45	2.44	2.45	21.42
102.0	0.0	19.4	6.71	2.45	2.44	2.45	21.64
103.0	0.0	19.75	6.86	2.45	2.44	2.45	21.85
104.0	0.0	20.1	7.01	2.45	2.44	2.45	22.06
105.0	0.0	20.46	7.16	2.45	2.44	2.45	22.27
106.0	0.0	20.82	7.32	2.45	2.44	2.45	22.48
107.0	0.0	21.18	7.47	2.45	2.44	2.45	22.7
108.0	0.0	21.55	7.62	2.45	2.44	2.45	22.91
109.0	0.0	21.92	7.78	2.45	2.44	2.45	23.12
110.0	0.0	22.29	7.94	2.45	2.44	2.45	23.33
111.0	0.0	22.67	8.09	2.45	2.44	2.45	23.55
112.0	0.0	23.05	8.26	2.45	2.44	2.45	23.76
113.0	0.0	23.43	8.42	2.45	2.44	2.45	23.97
114.0	0.0	23.82	8.58	2.45	2.44	2.45	24.18
115.0	0.0	24.21	8.74	2.45	2.44	2.45	24.39
116.0	0.0	24.6	8.91	2.45	2.44	2.45	24.61
117.0	0.0	25.0	9.08	2.45	2.44	2.45	24.82
118.0	0.0	25.4	9.25	2.45	2.44	2.45	25.03
119.0	0.0	25.8	9.42	2.45	2.44	2.45	25.24
120.0	0.0	26.21	9.59	2.45	2.44	2.45	25.45

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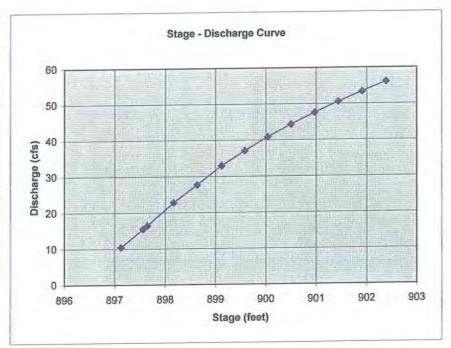


Q = 2/3 C L [ (2 g) ^ 0.5 ] (H1 - H2) ^ 1.5 + C L H2 [ 2 g ( H1 - H2) ] ^ 0.5

Reference; "The Engineers' Manual", Ralph G. Hudson, 2nd Edition, p 149-150.

### Calculated Stage-Discharge Data

Stage	Discharge
(feet)	(cfs)
902.38	56.00
901.91	53.28
901.44	50.47
900.97	47.47
900.50	44.27
900.04	40.86
899.58	37.08
899.11	32.91
898.63	27.67
898.16	22.75
897.63	16.38
897.55	15.45
897.12	10.38



## Main Ash Pond - Phase 1 Riser Discharge Rating Curve Calculations

Embankment Crest Elev.	802.0	feet
Riser Spillway Outlet Elev.	895.0	feet
Emergency Spillway Elev.	N/A	feet
Phase 1 Top Stoplog Elev.	896.0	feet
Outlet Pipe Diameter	2.45	feet
Stoplog Width	3.0	feet
Top of Stoplog Gate Elev.	N/A	feet
Gate Width	N/A	feet
Top of Riser Opening Elev.	N/A	feet

Phase 1 Normal Pool = 897.55 | feet

		_
		-
	,	>
		-
		-
096	959 958 957 926	1

	Phase 1 Normal Pool Elev, 897,557	Top Stoplog Elev. 896.0'	Secondary Spillway Pipe	
	Principal Spillway Pibe			
959 958 957 926	899 898	896	893.5	891.5

Stage (feet)	Weir Flow Over Stoplog	ver Stoplog Q <sub>1</sub>	Stage in Riser (feet)	30" RCP Outlet Pipe H Q <sub>2</sub>	outlet Pipe Q2
902.38	6.38	56.00	902.00	7.00	56.00
901.91	5.91	53,28	901.50	6.50	53,28
901.44	5,44	50.47	901.00	6,00	50.47
900.97	4.97		900.50	5.50	47.47
900.50	4.50	44.27	900.00	5.00	44.27
900.04	4.04	40.86	899.50	4.50	40.86
899.58	3,58	37.08	899.00	4.00	37.08
899.11	3,11	32.91	898.50	3.50	32.91
898.63	2.63	27.67	898.00		27.67
898,16	2,16	22.75	897,50	2.50	
897.63	1.63	16.38	897.00	2.00	16.38
897.55	1.55	15.45	896.92	1.92	A
897.12	1.12	10.38	896.50	1.50	10.38
896.00	00.00	00.00	895.00	0.00	0.00

<sup>1</sup> - Flow calculated from Submerged Weir Eqn.
<sup>2</sup> - Pipe Rating Curve from HydroCalc

otage	Area	Discharge
(E)	(acre)	(cfs)
96.00	72.36	0.0
97.12	73.15	10,4
897,55	73.45	15.5
897.63	73.50	16.4
898.16	73.87	22.8
898.63	74.19	27.7
899,11	74.52	32.9
899.58	74.84	37.1
900.04	75.15	40.9
900.50	75,45	44.3
26.006	75.75	47.5
901.44	76.06	50.5
16,106	76.36	53.3
902.38	78.67	56.0

## Main Ash Pond - Phase 2 Riser Discharge Rating Curve Calculations

Riser Spillway Outlet Elev. 895.0  Emergency Spillway Elev. N/A Phase 2 Top Stoplog Elev. 908.5  Outlet Pioe Diameter 2 45	feet
Ш	
	feet
-	feet
	feet
Stoplog Width 3.0	feet
Top of Stoplog Gate Elev. N/A	feet
	feet
op of Riser Opening Elev. N/A	feet

		Phase 2 Normal Pool Elev. 907.90
		5
096	959 958 957	606

Control

Weir Flow Over Stoplog H O<sub>2</sub>

30" RCP Outlet Pipe H Q,

Stage (feet)

Pipe Pipe Weir Weir Weir Weir Weir Weir Weir

36.8 26.3 177.1 15.4 9.3 3.3

85.07 83.34 81.57 81.43 79.74 77.88

154.1 136.7 120.0 104.0 88.8 74.4 60.9

97.91 96.39 94.85 93.28 91.71 90.10 88.47

913.00 912.00 911.50 910.50 909.50 909.50	907.50 907.50 907.00 906.50	1 - Rating C 2 , Q = CLP Qps = Prin	G
	Phase 2 Normal Pool Elev. 907,90'		
		Top Stoplog Elev. 906.5'	

206 906 905

$^1$ - Rating Curve from HydroCalc $^2$ - Q = CLH $^{1.5}$ Where C = 3.1, L = 3. Qpg = Principal Spillway Total Flow	Diecharos
Curve from H <sup>1,5</sup> Wher ncipal Spilly	Area
1 - Rating 2 - Q = CL Qps = Prir	Stade

Discharge	(cfs)	0.0	3.3	15.5	15,4	17.1	26.3	36.8	48.3	60'9	74.4	88.8	93.3	94.8	96,4	97.9
Area	(acre)	79.30	79.60	79.90	80.14	80.20	80.48	80.75	81.03	81.30	81.55	81.80	82.05	82.30	82.55	82.79
afige	€	906.5	907.0	907.5	6.706	908.0	908.5	908.0	908.5	910.0	910.5		911.5		912.5	

Secondary Spillway Pipe

> 893.5 892.5 891.5

Principal Spillway Pipe

868 897 896 895

# Main Ash Pond - Phase 3 Riser Discharge Rating Curve Calculations

Riser Spillway Outlet Elev, 895.0 Emergency Spillway Elev. N/A Phase 3 Top Stoplog Elev. 923.0 Outlet Pipe Diameter 2.45 Stoplog Width	feet feet feet feet	9 8 8 6 6
by Spillway Elev.  Op Stoplog Elev.  e Diameter	feet feet feet	960
op Stoplog Elev.	feet	959
e Diameter	feet	959
e Diameter	feet	1
Stoplog Width	foot	0
1000	IGGI	828
Top of Stoplog Gate Elev. N/A	feet	
Gate Width N/A	feet	957
op of Riser Opening Elev. N/A	feet	
Dhace 3 Normal Dool =	- Cont	

	Phase 3 Normal Pool Elev. 924.40'		30" Diameter Secondary Spillway Pipe
	Top Stoplog	30" Principal Spillway Pipe	
9 9 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8	926 924 1 924 923 922	898 768 896 896	892.5

Control		Weir											
Composite	S S	104.0	88.8	74.4	609	48.3	36.8	26.3	17.1	15.4	0	60	00.0
ver Stoplog	K2	104.0	88.8	74.4	60.9	48.3	36.8		17.1	15.4	9.3	60	0.0
Weir Flow Over Stoplog		5.0	4.5	4.0	3.5	3.0	2.5	2.0	1.5	4.7	1.0	0.5	0.0
30" RCP Outlet Pipe		135.6	134.5	133.4	132.3	131.2	130.0	128.9	127.8	127.1	126.6	125.5	124.3
30" RCP O		33,0	32.5	32.0	31.5	31.0	30.5	30.0	29.5	29.4	29.0	28.5	28.0
Stage (feet)	10001	928.00	927.50	927.00	926.50	926.00	925.50	925.00	924.50	924.40	924.00	923.50	923.00

1 - Rating Curve from HydroCalc
2 - Q = CLH<sup>1,3</sup> Where C = 3.1, L = 3' Q<sub>PS</sub> = Principal Spillway Total Flow

Discharge	(cfs)	0.0	3.3	9.3	15.4	17.1	26.3	36.8	48.3	60.9	74.4	88.8	104.0
Nega Nega	(acre)	87.78	88.04	88.30	88.50	88,55	88.80	89.06	89.31	89.57	89.82	80'08	90,33
afino	(ft)	923.0	923.5	924.0	924.4	924.5	925.0	925,5	926.0	926.5	927.0	927.5	928.0

# Main Ash Pond - Phase 4 Riser Discharge Rating Curve Calculations

									L	<
961		960		858		958		1967		
feet	feet	feet	feet	2.3.3	feet	feet	feet	feet	feet	feet
946.0	895.0	A/A	941.0		2.45	3.0	N/A	N/A	N/A	942.40
Embankment Crest Elev.	Riser Spillway Outlet Elev.	Emergency Spillway Elev.	Phase 4 Top Stoplog Elev.		Outlet Pipe Diameter	Stoplog Width	Top of Stoplog Gate Elev.	Gate Width	Top of Riser Opening Elev.	Phase 4 Normal Pool =

	Phase 4 Normal Pool Elev. 942,40'		
		Top Stoplog Elev. 941.0'	
958 858 858 957	944	942	940

15.4

47.0 46.5

941.5

942.5 942.4 942.0

74.4 60.9 48.3 36.8 26.3

104.0 888.8 74.4 60.9 48.3 36.8 26.3

170.1 169.2 167.5 166.6

165.7 164.9 164.0 163.1 162.2

48.5 48.0 47.5

944,5 944.0 943.5

49.6

Weir Flow Over Stoplog

30" RCP Outlet Pipe H Q<sub>1</sub>

Stage (feet) 946.0 945.0

				30" Diameter	Spillway Pipe			
	30,	Spillway						
898	897	896	895	894		892.5	n +00	031.0

n HydroCalc	Where C = 3.1, L = 3'	ray Total Flow
Rating Curve from	10	ncipal Spillw
- Rating	O = CLH	PS = Prir

Discharge	(cfs)	0.0	3.3	9.3	15.4	17.1	26.3	36.8	48.3	60.9	74.4	88.8	104.0
Area	(acre)	97.12	97.38	97.65	97.87	97.92	98.19	98.46	98.72	98.99	99.26	99.53	99.80
Stage	(#)	941.0	941.5	942.0	942.4	942.5	943.0	943.5	944.0	944.5	945.0	945.5	948.0

Main Ash Pond - Phase 5 Riser Discharge Rating Curve Calculations

		I	Normal Pool		111111111111111111111111111111111111111		change			
961		008	959		Top Stoplog	957 Elev. 957.0°	THE PROPERTY OF THE PARTY OF TH	926	955	
feet	feet	1991	Teet	feet	feet	feet	feet	feet	feet	
962.0	885.0	N/M	0.748	2.5	4.0	957.0	4.0	960.0	958.16	
Embankment Crest Elev.	Emergency Collins Class	Dhase & Top Storlog Elev.	riese o 10p otopiog Elev.	Outlet Pipe Diameter	Stoplog Width	Top of Stoplog Gate Elev.	Gate Width	Top of Riser Opening Elev.	Phase 5 Normal Pool =	

NO.	ő	Over Gate Orifice Flow	De Weir Flow Over Gate Orifice Flow
63		5.0 138.6 3	5.0 138.6
3.00			
CA			4.0 99,2
			3.0 64.4
	49.0	2.5 49.0	2,5
	35.1	2.0 35.1	191.2 2.0 35.1
	22.8	1.5 22.8	190,4 1.5 22.8
	15,5	1,2 15,5	190,0 1,2 15,5
	12.4	1.0 12.4	189.7 1.0 12.4
	4.4	0.5 4.4	188.9 0.5 4.4
	000	00	188.1

 $<sup>^1</sup>$  - Rating Curve from HydroCalc  $^2$  - Q = CLH  $^{1.5}$  Where C = 3.1, L = 4'  $^3$  - Q = CA(2gH)  $^{0.6}$  Where C = 0.61, A = 12, g = 32.17 Gps = Principal Spillway Total Flow

Discharge	(cls)	0.0	4.4	12.4	15.5	22.8	35.1	49.0	64.4	81.2	92.8	101.7	109.8
Area	(acre)	105.78	106.06	106.33	106.42	106.61	106.88	107.15	107.43	107.71	107.99	108,28	108.56
Stage	(H)	957.0	857.5	958.0	958.2	958.5	959.0	59	960.0	960.5	961.0	961.5	962.0

30" Principal Spillway Pipe

897 898 895 894

30" Diameter Secondary Spillway Pipe

892.5 891.5

EWB\_Main\_P1\_100YR.OUT

SITES	01/01/200	SEWB1	EW BROWN	MAIN POND	PHASE	1 100YR	0.1595	3
STRUCTURE	EWB1	EW BROWN						
		897.55	73.45	15.5				
-		897.63	73.51	16.4				
		898.16	73.87	22.8				
		898.63	74.19	27.7				
		899.11	74.52	32.9				
		899.58	74.84	37.1				
		900.04	75.15	40.9				
		900.50	75.45	44.3				
		900.97	75.75	47.5				
		901.44	76.06	50.5				
		901.91	76.36	53.3				
		902.38	76.67	56.0	1000			
ENDTABLE								
POOLDATA	ELEV	897.55	897.55					
WSDATA	5C 1 2 AC	95	102.1	0.25				
STORM	30 2 2 110		6			97.	.2	
	QSNL		4.53					
ENDJOB	QUIL.							

\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 4.53	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	CN 95.00	DA-SM 0.16	TC/L 0.25	-/н 0.00	QRF 0.00
SITEDATA-	PERM POOL 897.55	CREST PS 897.55	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 97.20	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND	COND L	DIA/W Page 1	-/H	

	0.00	0.00	B_Main_P	L_100YR.0	UT 0.00		
	PS N 0.000	0.00		WEIR L 0.00	TW EL 0.00		
	2ND STG 0.00	ORF H 0.00		ORF L 0.00	START AUX	Χ.	
ASCRESTS	- AUX.1 0.00	AUX.2 0.00		0.00	AUX.4 0.00		AUX.5 ).00
AUX.DATA	REF.NO.	RETARD. Ci 0.00		TATION 0.00	INLET LENGT 0	ГН	
AUX.DATA	0.000	SIDE SLOPE 0.00	E)	XIT N 0.000	EXIT SLOPE 0.000		
BTM WIDTH	- BW1 0.00	BW2 0.00		BW3 0.00	BW4 0.00	(	BW5 0.00
	/2007 EW E 0.1 4:38					SUBW= 1	
ROUTING OF	STORM HYDRO	GRAPH START:	S AT ELEV	ATION 8	397.55		
PERM POOL	897.55	FT 0	.0 ACFT	73.45	AC 15.	5 CFS	
CREST PS	897.55	FT 0	.0 ACFT	73.45	AC 15.	5 CFS	
SED ACCUM	897.55	FT 0	.0 ACFT	73.45	AC 15.	5 CFS	
BASEFLOW	897.55	FT 0.	.0 ACFT	73.45	AC 15.	5 CFS	
RATING TAB	*********** LE DEVELOPED ND AUX. GIVE	, SITE = EWE	31:		****	*****	****
ELE FEE 1 897. 2 897. 3 898. 4 898. 5 899. 6 899. 7 900. 8 900. 9 900. 10 901. 11 901. 12 902.	T CFS 55 15.50 63 16.40 16 22.80 63 27.70 11 32.90 58 37.10 04 40.90 60 44.30 97 47.50 44 50.50 91 53.30 1056.00	Q-PS CFS 15.50 16.40 22.80 27.70 32.90 37.10 40.90 44.30 47.50 50.50 53.30 56.00	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	AC-F 0.0 5.8 44.9 79.7 115.4 150.5 185.0 219.6 255.1 290.8 326.6 362.6	ACRE 73.45 8 73.51 3 73.87 3 74.19 2 74.52 2 74.84 2 75.15 5 75.45 8 75.75 6 76.06 8 76.36 76.67		
	D= 6.00 TC= 0.25	HR CN=		VOL=		DA= 0.16	SM
FCAR =	- 344.0	CF3, AI	Page				

### EWB\_Main\_P1\_100YR.OUT

X CFS
ONS.
NS.
SM
****

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 897.55

ROUTED BTM WIDTH MAX ELEV VOL-MAX AREA-MAX AUX.-HP VOL-AUX.
RESULTS FT FT ACFT AC FT ACFT ACFT AC FT 32.2 73.8 0.00 ACFT 0.0 FT FT 0.0 897.99 STORM HYD

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

ISITES....JOB NO. 1 COMPLETE.

EWB1 EW BROWN MAIN POND PHASE 1 100YR 0.1595 Page 3

### EWB\_Main\_P1\_100YR.OUT

- O SUBWATERSHED(S) ANALYZED.
  - 1 STRUCTURE(S) ANALYZED.
  - 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
  - O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

SITES....COMPUTATIONS COMPLETE

1				SUMMARY	TABLE	1			2005.0.1 /01/2005
WATE	RSHED ID			RU	N DATE				RUN TIME
EWB1				10/1	8/2007				09:44:38
>>>	SITE	SUBWS	SUBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC	
	EWB1	1	0.16	95.	0.25	0.16	TR60	C	
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CRES ELEV (FT)	T BTM. WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)		ST.	(IT* /EL. /SEC)	TYPE HYD
1	0.0	901.9	0.0	-3.9	898.0	0.	0.	0.0 S	TORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

 EWB\_Main\_P1\_PMP.OUT

SITES STRUCTURE	01/01/200 EWB1	5EWB1 EW BROWN 897.55 897.63 898.16 898.63 899.11 899.58 900.04 900.50	EW BROWN MAIN POND 73.45 73.51 73.87 74.19 74.52 74.84 75.15 75.45	MAIN POND 15.5 16.4 22.8 27.7 32.9 37.1 40.9 44.3	PHASE 2	2 PMP	0.1595	3	
		900.30 900.97 901.44 901.91 902.38	75.75 76.06 76.36 76.67	47.5 50.5 53.3 56.0	1000				
ENDTABLE POOLDATA WSDATA STORM GO,RAINS ENDJOB	ELEV 5C 1 2 AC QSNL	897.55 95	897.55 102.1 6 28	0.25		9	7.2		

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

\*\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

1STTES					
XEQ 10/18/2007	EW BROWN	MAIN POND PHASE	2 PMP		WSID= EWB1
VER 2005.0.1	EW	BROWN MAIN POND			SUBW= 1
TIME 10:00:18	SITE =	EWB1	PASS=	1	PART= 1

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 28.00	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	CN 95.00	DA-SM 0.16	TC/L 0.25	-/H 0.00	QRF 0.00
SITEDATA-	PERM POOL 897.55	CREST PS 897.55	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 97.20	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND	COND L	DIA/W Page 1	-/H	

0.00	0.00	_Main_P1_PMP.OU 0.00	0.00	
0.00		0.00	0.00	
PS N 0.000		WEIR L 0.00		
2ND STG 0.00	ORF H 0.00	ORF L 0.00	START AUX. 0.00	
ASCRESTS - AUX.1		AUX.3 0.00	AUX.4 0.00	AUX.5 0.00
AUX.DATA - REF.NO.		TIE STATION 0.00	INLET LENGTH	
AUX.DATA - INLET N 0.000	SIDE SLOPE 0.00		EXIT SLOPE 0.000	
BTM WIDTH - BW1 0.00		BW3 0.00	BW4 0.00	BW5 0.00
XEQ 10/18/2007 VER 2005.0.1 TIME 10:00:18	EW BROWN MAI EW BRO SITE = EWB	N POND PHASE 2 WN MAIN POND 1	PMP PASS= 1	WSID= EWB1 SUBW= 1 PART= 2
				.,,,,
ROUTING OF STORM HYD	RUGRAPH STAKIS	AT ELEVATION	897.55	
PERM POOL 897.	55 FT 0.0	ACFT 73.45	AC 15.5	CFS
CREST PS 897.	55 FT 0.0	ACFT 73.45	AC 15.5	CFS
SED ACCUM 897.	55 FT 0.0	ACFT 73.45	AC 15.5	CFS
BASEFLOW 897.	55 FT 0.0	ACFT 73.45	AC 15.5	CFS
******	*****	******	*****	******
RATING TABLE DEVELOP WITH PS AND AUX. GI				
RATING TABLE NUMBER ELEV. Q-TO FEET CF 1 897.55 15.	TAL Q-PS S CFS	Q-AUX. VOLUI CFS AC-	FT ACRE	

RATIN	IG TABLE	NUMBER 2					
	ELEV.			Q-AUX.			
		CFS					
1		15.50			0.00	73.45	
2	897.63	16.40	16.40	0.00	5.88	73.51	
3	898.16	22.80	22.80	0.00	44.93	73.87	
4		27.70			79.73	74.19	
5	899.11	32.90	32.90	0.00	115.42	74.52	
	899.58	37.10				74.84	
7	900.04	40.90	40.90			75.15	
	900.50	44.30	44.30	0.00	219.65	75.45	
		47.50		0.00	255.18	75.75	
10	901.44	50.50	50.50	0.00	290.86	76.06	
11		53.30			326.68		
12	902.38	1056.00	56.00	1000.00	362.64	76.67	
STORM	HYD					IN DA=	0.16 SM
		TC=0.25	HR CN=	95.00	VOL= 33	5.4 ACFT	
- 3	PEAK =	2158.6	CFS, AT	2.5 HRS.			

PEAK = 2158.6 CFS, AT 2.5 HRS. Page 2

## EWB\_Main\_P1\_PMP.OUT

ROUTED RESULT	HYD TYPE STORM HYD	EMAX 900.56 FT 2	VOL-MAX 224.0 ACFT	AMAX 75.49 AC	QMAX 44.7 CFS
PS STORAGE	224.0 AC	FT, BETWEEN AU	IX. CREST AND	SED. ACCUM E	LEVATIONS.
PERM POOL	897.55 FT	0.0 ACFT	73.45 AC	15.5 CFS	
CREST PS	897.55 FT	0.0 ACFT	73.45 AC.	15.5 CFS	
SED ACCUM	897.55 FT	0.0 ACFT	73.45 AC	15.5 CFS	
BASEFLOW	897.55 FT	0.0 ACFT	73.45 AC	15.5 CFS	
AUX. CREST	901.91 FT	326.7 ACFT	76.36 AC	53.3 CFS	
PS STORAGE	326.7 ACF	T, BETWEEN AUX	. CREST AND S	ED. ACCUM EL	EVATIONS.
START ELEV	897.55 FT	0.0 ACFT	73.45 AC	15.5 CFS	
STORM HYD C	0= 6.00 HR = 0.25 HR	P= 28.00 IN CN= 95.00	Q= 27.38 I VOL= 335.	N DA= 4 ACFT	0.16 SM
PEAK =	2158.6 CFS,	AT 2.5 HRS.			

WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RATI	NG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	897.55	15.50	15.50	0.00	0.00	73.45
2	897.63	16.40	16.40	0.00	5.88	73.51
3	898.16	22.80	22.80	0.00	44.93	73.87
4	898.63	27.70	27.70	0.00	79.73	74.19
5	899.11	32.90	32.90	0.00	115.42	74.52
6	899.58	37.10	37.10	0.00	150.52	74.84
7	900.04	40.90	40.90	0.00	185.02	75.15
8	900.50	44.30	44.30	0.00	219.65	75.45
9	900.97	47.50	47.50	0.00	255.18	75.75
10	901.44	50.50	50.50	0.00	290.86	76.06
11	901.91	53.30	53.30	0.00	326.68	76.36
12	902,38	1056.00	56.00	1000.00	362.64	76.67

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 897.55

ROUTED	BTM WIDTH	MAX ELEV	VOL-MAX	AREA-MAX	AUXHP	VOL-AUX.
RESULTS	FT	FT	ACFT	AC	FT	ACFT
STORM HYD	0.0	900.56	224.0	75.5	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

1SITES....JOB NO. 1 COMPLETE.

EWB1 EW BROWN MAIN POND PHASE 2 PMP Page 3

### EWB\_Main\_Pl\_PMP.OUT

- O SUBWATERSHED(S) ANALYZED.
- 1 STRUCTURE(S) ANALYZED.
- 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
- O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

SITES....COMPUTATIONS COMPLETE

1 SUMMARY TABLE 1 SITES VERSION 2005.0.1 DATED 01/01/2005

WATER	RSHED ID			RU	N DATE				RUN TIME
EWB1				10/1	8/2007				10:00:18
>>>	SITE	SUBWS ID	SUBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRU CLAS	=
	EWB1	1	0.16	95.	0.25	0.16	TR60	C	
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CREST ELEV (FT)	WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)	VOL. DI	ST.	EXIT* VEL. T/SEC)	TYPE HYD
1	0.0	901.9	0.0	-1.4	900.6	0.	0.	0.0	STORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

EWB\_Main\_P2\_100YR.OUT 1\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\* SITES XEQ 10/18/2007 WATER RESOURCE SITE ANALYSIS COMPUTER PROGRAM VER 2005.0.1 (USER MANUAL - DATED MAY 2001) TIME 09:44:48 \*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\* 80-80 LIST OF INPUT DATA \*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\* EW BROWN MAIN POND PHASE 2 100YR 0.1603 3 01/01/2005EWB1 EW BROWN MAIN POND STRUCTURE EWB1 907.90 80.14 15.45 908.0 80.20 17.1 80.48 26.3 908.5 80.75 36.8 909.0 909.5 48.3 81.03 60.9 81.30 910.0 81.55 910.5 74.4 88.8 911.0 81.80 82.05 911.5 93.3 82.30 912.0 94.8 912.5 82.55 96.4 97.9 913.0 82.79 1000 ENDTABLE 907.90 POOLDATA ELEV 907.90 102.6 5C 1 2 AC 95 WSDATA 0.25 96.37 6 STORM GO, RAINS QSNL 4.53 **ENDJOB** \*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\* \*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES. \*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK. 1SITES -----XEQ 10/18/2007 EW BROWN MAIN POND PHASE 2 100YR 0.1603 WSID= EWB1 VER 2005.0.1 EW BROWN MAIN POND SUBW= 1 PASS= 1 PART= 1 TIME 09:44:48 SITE = EWB1\*\*\*\*\*\*\* BASIC DATA \*\*\*\*\*\*\* CLIMATE AREA - NOT DEFINED DESIGN CLASS C STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 4.53	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	CN 95.00	DA-SM 0.16	TC/L 0.25	-/н 0.00	QRF 0.00
SITEDATA-	PERM POOL 907.90	CREST PS 907.90	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 96.37	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND	COND L	DIA/W Page 1	-/H	

		0.00		0.00		Main_P2_	_100YR.0	DUT	0.00		
		PS N 0.000		0.00			EIR L 0.00		TW EL 0.00		
	- 7	2ND STG 0.00		ORF 1			ORF L	ST	ART AUX	۲.	
ASCR	ESTS -	AUX.1 0.00		AUX.2			UX.3 0.00		AUX.4 0.00		AUX.5 0.00
AUX.	DATA -	REF.NO.	RE	TARD. 0		TIE ST	ATION 0.00	INLE	T LENGT		
AUX.	DATA - I	0.000	SII	DE SLOF 0.00		EX:	IT N .000	EXI	SLOPE 0.000		ACTUAL AUX NO
BTM V	WIDTH -	BW1 0.00		BW2			BW3		BW4 0.00		BW5 0.00
TIME	2005.0.1 09:44:4 ING OF S	8 TORM HYDRO		SITE =	EWB			PASS=	1		BW= 1 RT= 2
	POOL S	907.90				ACFT				4 CFS	
CREST		907.90				ACFT			15.4		
SED A	ACCUM	907.90	FT		0.0	ACFT	80.14	AC	15.4	4 CFS	
BASEF	FLOW	907.90	FT		0.0	ACFT	80.14	AC	15.4	4 CFS	
RATIN WITH	NG TABLE I PS AND NG TABLE ELEV.	DEVELOPED AUX. GIVE NUMBER 2 Q-TOTA	, SI N -	TE = EI NO ASD	WB1 ATA	: RECORD	GIVEN.	ME	AREA	***	****
1 2 3 4 5 6	FEET 907.90 908.00 908.50 909.00 909.50 910.00 910.50 911.00	15.45 17.10 26.30 36.80 48.30 60.90 74.40 88.80		CFS 15.45 17.10 26.30 36.80 48.30 60.90 74.40 88.80 93.30		0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	169.5 210.2 251.0 292.0	00 02 19 49 94 52 23 07	ACRE 80.14 80.20 80.48 80.75 81.03 81.30 81.55 81.80 82.05		
7 8 9 10 11 12	911.50 912.00 912.50 913.00	93.30 94.80 96.40 1097.90		94.80 96.40 97.90	1	0.00 0.00 000.00	333.1 374.3 415.6	33	82.30 82.55 82.79		

PEAK = 346.2 CFS, AT 2.5 HRS. Page 2

## EWB\_Main\_P2\_100YR.OUT

ROUTED RESUL	T - HYD TYPE STORM HYD	EMAX 908.30 FT	VOL-MAX 31.8 ACFT	AMAX 80.37 AC	QMAX 22.6 CFS
PS STOR	AGE 31.8 A	CFT, BETWEEN	AUX. CREST AND	SED. ACCUM E	ELEVATIONS.
PERM POOL	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
CREST PS	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
SED ACCUM	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
BASEFLOW	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
AUX. CREST	912.50 FT	374.3 ACFT	82.55 AC	96.4 CFS	
PS STOR	AGE 374.3 ACE	T, BETWEEN AL	JX. CREST AND SI	ED. ACCUM EL	EVATIONS.
START ELEV	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
STORM HYD	D= 6.00 HR TC= 0.25 HR	P= 4.53 IN CN= 95.00	Q= 3.95 IN VOL= 135.9	DA=	0.16 SM
PEAK =	346.2 CFS,	AT 2.5 HRS			
****	*****	******	*****	******	*****
	DEVELOPED, SITE AUX. GIVEN - NO		GIVEN.		

RATIN	NG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	907.90	15.45	15.45	0.00	0.00	80.14
2	908.00	17.10	17.10	0.00	8.02	80.20
3	908.50	26.30	26.30	0.00	48.19	80.48
4	909.00	36.80	36.80	0.00	88.49	80.75
5	909.50	48.30	48.30	0.00	128.94	81.03
6	910.00	60.90	60.90	0.00	169.52	81.30
7	910.50	74.40	74.40	0.00	210.23	81.55
8	911.00	88.80	88.80	0.00	251.07	81.80
9	911.50	93.30	93.30	0.00	292.03	82.05
10	912.00	94.80	94.80	0.00	333.12	82.30
11	912.50	96.40	96.40	0.00	374.33	82.55
12	913.00	1097.90	97.90	1000.00	415.67	82.79

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 907.90

ROUTED	BTM WIDTH	MAX ELEV	VOL-MAX	AREA-MAX	AUXHP	VOL-AUX.
RESULTS	FT	FT	ACFT	AC	FT	ACFT
STORM HYD	0.0	908.30	31.8	80.4	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

1SITES....JOB NO. 1 COMPLETE.

### EWB\_Main\_P2\_100YR.OUT

- O SUBWATERSHED(S) ANALYZED.
- 1 STRUCTURE(S) ANALYZED.
- 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
- O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

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### SITES....COMPUTATIONS COMPLETE

1				SUMMARY	TABLE	1	SITES V	ERSION 2 TED 01/0	
WATER	RSHED ID	F.		RU	N DATE			R	UN TIME
EWB1				10/1	8/2007			0	9:44:48
>>>	SITE	SUBWS S	SUBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC CLASS	<<<
	EWB1	1	0.16	95.	0.25	0.16	TR60	C	
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CREST ELEV (FT)	WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)		ST. V	(IT* /EL. /SEC)	TYPE HYD
1	0.0	912.5	0.0	-4.2	908.3	0.	0.	0.0 ST	ORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

EWB\_Main\_P2\_PMP.OUT

TIME 09:44:50

SITES	01/01/200	5EWB1	EW BROWN	MAIN POND	PHASE	2 PMP	0.1603	3	
STRUCTURE		EW BROWN	MAIN POND						
		907.90	80.14	15.45					
		908.0	80.20	17.1					
		908.5	80.48	26.3					
		909.0	80.75	36.8					
		909.5	81.03	48.3					
		910.0	81.30	60.9					
		910.5	81.55	74.4					
		911.0	81.80	88.8					
		911.5	82.05	93.3					
		912.0	82.30	94.8					
		912.5	82.55	96.4					
		913.0	82.79	97.9	1000				
ENDTABLE									
POOLDATA	ELEV	907.90	907.90						
WSDATA	5C 1 2 AC		102.6	0.25					
STORM			6			9	6.37		
GO, RAINS	OSNL		28						
ENDJOB									

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

1SITES			
XEQ 10/18/2007	EW BROWN MAIN POND PHASE 2 PMP		WSID= EWB1
VER 2005.0.1	EW BROWN MAIN POND		SUBW= 1
TIME 09:44:50	SITE = EWB1 PASS=	1	PART= 1

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 28.00	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	CN 95.00	DA-SM 0.16	TC/L 0.25	-/H 0.00	QRF 0.00
SITEDATA-	PERM POOL 907.90	CREST PS 907.90	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 96.37	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND	COND L	DIA/W Page 1	-/H	

				EWE	Main	P2_PMP.O	UT			
		0.00	0	.00		0.00	01	0.00		
		PS N 0.000		KE .00		WEIR L 0.00		TW EL 0.00		
	21	ND STG 0.00	OR 0	F H		ORF L 0.00	ST	ART AUX		
ASCE	RESTS -	AUX.1 0.00	AUX 0	x.2		AUX.3 0.00		AUX.4 0.00		AUX.5 0.00
AUX.	DATA - F	REF.NO.	RETARD 0		TIE S	O.00	INLE	T LENGTH	L	
AUX.	DATA - IN	0.000	SIDE SI	OPE 00		0.000	EXI	T SLOPE 0.000	ACT	UAL AUX? NO
втм	WIDTH -	BW1 0.00		8W2 .00		BW3 0.00		BW4 0.00		BW5 0.00
XEQ VER TIME	10/18/200 2005.0.1 09:44:50		SITE	W BRO = EWB	WN MAI		PASS=		WSID= SUBW= PART=	1
		907.90				80.14			CFS	
CRES	T PS	907.90	FT	0.0	ACFT	80.14	4 AC	15.4	CFS	
SED .	ACCUM	907.90	FT	0.0	ACFT	80.14	4 AC	15.4	CFS	
BASE	FLOW	907.90	FT	0.0	ACFT	80.14	AC AC	15.4	CFS	
RATI	NG TABLE	******** DEVELOPED, AUX. GIVEN	SITE =	EWB1				*****	*****	*****
1 2 3 4 5 6 7	NG TABLE I ELEV. FEET 907.90 908.00 908.50 909.00 909.50 910.00 910.00	Q-TOTAL CFS 15.45 17.10 26.30 36.80 48.30 60.90 74.40	Q-P: CF: 15. 17. 26. 36. 48. 60. 74.	5 45 10 30 30 30 30 40	Q-AUX CFS 0.00 0.00 0.00 0.00	AC- 0 0. 0 8. 0 48. 0 88. 0 128. 0 169. 0 210.	FT 00 02 19 49 94 52 23	AREA ACRE 80.14 80.20 80.48 80.75 81.03 81.30 81.55		

STORM HYD D= 6.00 HR P= 28.00 IN Q= 27.38 IN DA= 0.16 SM TC= 0.25 HR CN=  $95.00 \cdot$  VOL= 336.2 ACFT

0.00

0.00

0.00

96.40 0.00 374.33 97.90 1000.00 415.67

251.07

292.03

333.12

374.33

81.80

82.05

82.30

82.55

82.79

PEAK = 2169.1 CFS, AT2.5 HRS. Page 2

88.80

93.30

94.80

88.80

93.30

94.80

96.40

1097.90

8

9

10

11

12

911.00

911.50

912.00

912.50 913.00

### EWB Main P2 PMP.OUT

ROUTED RESULT -	HYD TYPE EN STORM HYD 910	MAX V .58 FT 21	0-	AMAX 81.59 AC	QMAX 76.8 CFS
PS STORAGE	216.9 ACFT,	BETWEEN AUX	. CREST AND S	ED. ACCUM EL	EVATIONS.
PERM POOL	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
CREST PS	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
SED ACCUM	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
BASEFLOW	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
AUX. CREST	912.50 FT 37	74.3 ACFT	82.55 AC	96.4 CFS	
PS STORAGE	374.3 ACFT,	BETWEEN AUX.	CREST AND SE	D. ACCUM ELE	VATIONS.
START ELEV	907.90 FT	0.0 ACFT	80.14 AC	15.4 CFS	
STORM HYD D	= 6.00 HR P= = 0.25 HR CN=	= 28.00 IN = 95.00	Q= 27.38 IN VOL= 336.2	DA=	0.16 SM
PEAK =	2169.1 CFS, AT	2.5 HRS.			
****	*****	******	*****	****	****

RATING TABLE DEVELOPED, SITE = EWB1 : WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RATI	NG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	907.90	15.45	15.45	0.00	0.00	80.14
2	908.00	17.10	17.10	0.00	8.02	80.20
3	908.50	26.30	26.30	0.00	48.19	80.48
4	909.00	36.80	36.80	0.00	88.49	80.75
5	909.50	48.30	48.30	0.00	128.94	81.03
6	910.00	60.90	60.90	0.00	169.52	81.30
7	910.50	74.40	74.40	0.00	210.23	81.55
8	911.00	88.80	88.80	0.00	251.07	81.80
9	911.50	93.30	93.30	0.00	292.03	82.05
10	912.00	94.80	94.80	0.00	333.12	82.30
11	912.50	96.40	96.40	0.00	374.33	82.55
12	913.00	1097.90	97.90	1000.00	415.67	82.79

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 907.90

ROUTED	BTM WIDTH	MAX ELEV	2.00	AREA-MAX	AUXHP	VOL-AUX.
RESULTS STORM HYD	0.0	910.58	216.9	81.6	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

PEAK - CFS Q-PS DISCHARGE = 76.8 Q-TOT. 76.8 Q-AUX. 0.0

1SITES....JOB NO. 1 COMPLETE.

EWB1 EW BROWN MAIN POND PHASE 2 PMP Page 3

### EWB\_Main\_P2\_PMP.OUT

- O SUBWATERSHED(S) ANALYZED.
- 1 STRUCTURE(S) ANALYZED.
- 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
- O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

SITES....COMPUTATIONS COMPLETE

1				SUMMARY	TABLE	1	SITES VERSION 2005.0. DATED 01/01/2005			
WATE	RSHED ID			RU	N DATE			1	RUN TIME	
EWB1				10/1	8/2007			(	9:44:50	
>>>	SITE	SUBWS	SUBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC CLASS	<<<	
	EWB1	1	0.16	95.	0.25	0.16	TR60	C		
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CRES ELEV (FT)	T BTM. WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)	VOL. DI	ST.	VEL.	TYPE HYD	

910.6

0.

0. 0.0 STORM HYD

0.0 - 1.9

SITES.....SUMMARY TABLE 1 COMPLETED.

912.5

1 0.0

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

EWB Main P3 100YR.OUT

SITES	01/01/200	5EWB1	EW BROWN	MAIN POND	PHASE 3	3 100YR	0.1480	3
STRUCTURE		EW BROWN	MAIN POND					
Dinociona		924.40	88.50	15.45				
		924.50	88.55	17.1				
		925.00	88.80	26.3				
		925.50	89.06	36.8				
		926.00	89.31	48.3				
		926.50	89.57	60.9				
		927.00	89.82	74.4				
		927.50	90.08	88.8				
		928.00	90.33	104.0	1000			
ENDTADI E		320.00	30.33	101.0	1000			
ENDTABLE	CLEV	924.40	924.40					
POOLDATA	ELEV		94.7	0.25				
WSDATA	5C 1 2 AC	93		0.25		10	4.4	
STORM			6			10	4.4	
GO, RAINS	QSNL		4.53					
ENDJOB								

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\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 4.53	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	CN 95.00	DA-SM 0.15	TC/L 0.25	-/H 0.00	QRF 0.00
SITEDATA-	PERM POOL 924.40	CREST PS 924.40	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 104.40	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND 0.00	COND L 0.00	DIA/W 0.00	-/н 0.00	
	PS N	KE	WEIR L Page 1	TW EL	

		0.000	0.0	WB_Main_P3 O	0.00		
	2	2ND STG 0.00			ORF L 0.00	START AUX. 0.00	
ASCRE	STS -	AUX.1 0.00	AUX.2	2 /	0.00	AUX.4 0.00	
AUX.D	ATA -	REF.NO.	RETARD. 0		TATION 0.00	INLET LENGTH	
AUX.D	ATA - I	0.000	SIDE SLOP 0.00	PE EX	(IT N 0.000	EXIT SLOPE 0.000	ACTUAL AU
BTM W	IDTH -	BW1 0.00	BW2 0.00		BW3 0.00	BW4 0.00	
			EW SITE =				SUBW= 1 PART= 2
PERM F	P00L	924.40	FT	0.0 ACFT	88.50 /	AC 15.4	CFS
CREST	PS	924.40	FT	0.0 ACFT	88.50	AC 15.4	CFS
SED AC	CUM	924.40	FT	0.0 ACFT	88.50 4	AC 15.4	CFS
BASEFL	_OW	924.40	FT (	0.0 ACFT	88.50 A	AC 15.4	CFS
RATING	PS AND TABLE ELEV. FEET	NUMBER 2 Q-TOTAL CFS 15.45 17.10 26.30	Q-PS CFS 15.45 17.10 26.30	Q-AUX. CFS 0.00 0.00 0.00	VOLUME AC-FT 0.00 8.85 53.19	ACRE 88.50 88.55 88.80	
5 6 7 8	925.50 926.00 926.50 927.00 927.50 928.00	36.80 48.30 60.90 74.40 88.80 1104.00	88.80		186.97 231.81 276.79	89.31 89.57 89.82 90.08	
STORM H	HYD	D= 6.00 TC= 0.25	HR P=	4.53 IN 95.00	Q= 3.9 VOL=	95 IN DA 133.3 ACFT	A= 0.15 SM
				2.5 HRS.			

PS STORAGE 29.6 ACFT, BETWEEN AUX. CREST AND SED. ACCUM ELEVATIONS. Page 2

CUID	Maria	72	TOOME	OUT
EWB	Md III	PO.	100YR	.001

PERM POOL	924.40 FT	0.0 ACFT	88.50 AC	15.4 CFS	
CREST PS	924.40 FT	0.0 ACFT	88.50 AC	15.4 CFS	
SED ACCUM	924.40 FT	0.0 ACFT	88.50 AC	15.4 CFS	
BASEFLOW	924.40 FT	0.0 ACFT	88.50 AC	15.4 CFS	
AUX. CREST	927.50 FT	276.8 ACFT	90.08 AC	88.8 CFS	
PS STORAGE	276.8 ACFT,	BETWEEN AUX.	CREST AND	SED. ACCUM ELEVATIONS.	
START ELEV	924.40 FT	0.0 ACFT	88.50 AC	15.4 CFS	
	5 00	- 1	0 2 05	TH DA- 0 15 CM	

D= 6.00 HR P= 4.53 IN Q= 3.95 IN TC= 0.25 HR CN= 95.00 VOL= 133.3 ACFT STORM HYD DA= 0.15 SM

PEAK = 320.7 CFS, AT 2.5 HRS.

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

RATING TABLE DEVELOPED, SITE = EWB1 :

WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RATIN	NG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	924.40	15.45	15.45	0.00	0.00	88.50
2	924.50	17.10	17.10	0.00	8.85	88.55
3	925.00	26.30	26.30	0.00	53.19	88.80
4	925.50	36.80	36.80	0.00	97.65	89.06
5	926.00	48.30	48.30	0.00	142.25	89.31
6	926.50	60.90	60.90	0.00	186.97	89.57
7	927.00	74.40	74.40	0.00	231.81	89.82
8	927.50	88.80	88.80	0.00	276.79	90.08
9	928.00	1104.00	104.00	1000.00	321.89	90.33

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 924.40

ROUTED	BTM WIDTH	MAX ELEV	VOL-MAX	AREA-MAX	AUXHP	VOL-AUX.
RESULTS	FT	FT	ACFT	AC	FT	ACFT
STORM HYD	0.0	924.73	29.6	88.7	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

Q-PS Q-AUX. Q-TOT. 21.4 0.0 21.4 PEAK - CFS DISCHARGE =

1SITES....JOB NO. 1 COMPLETE.

EW BROWN MAIN POND PHASE 3 100YR 0.1480 EWB1

- O SUBWATERSHED(S) ANALYZED.
  - 1 STRUCTURE(S) ANALYZED.
- 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
- O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

### SITES....COMPUTATIONS COMPLETE

1				SUMMARY	TABLE	1		ERSION 20 FED 01/0	
WATERS	HED ID			RU	N DATE			RI	JN TIME
EWB1				10/1	8/2007			09	9:44:53
>>>	SITE	SUBWS S	SUBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC CLASS	<<<
E	EWB1	1	0.15	95.	0.25	0.15	TR60	С	
NO. V	DIA./ WIDTH EN/FT)	AUX.CREST ELEV (FT)	BTM. WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)		ST. V	(IT* T 'EL. 'SEC)	YPE HYD
1	0.0	927.5	0.0	-2.8	924.7	0.	0.	0.0 STC	RM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

Teeesessan	****			n_P3_PMP.0	UT *******	****	*****	*****
VER 2	10/18/2007 2005.0.1 09:44:56	WATER RES	OURCE S SER MAN	SITE ANALYS	SIS COMPU ED MAY 20	TER PROG 01)	RAM	
*****	******	**** 80-80	LIST (	OF INPUT DA	ATA ****	*****	*****	******
	01/01/2005EV			MAIN POND	PHASE 3	PMP 0.1	480	3
STRUCTURE	92 92 92 92 92 92 92 92	4.40 88 4.50 88 5.00 88 5.50 89 6.00 89 7.00 89 7.50 90	N POND .50 .55 .80 .06 .31 .57 .82 .08	15.45 17.1 26.3 36.8 48.3 60.9 74.4 88.8 104.0	1000			

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

0.25

104.4

\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

94.7

28

924.40

5C 1 2 AC 95

924.40

\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

1STTES					
XEQ 10/18/2007	EW BROWN MAIN POND PHAS	E 3	PMP		WSID= EWB1
VER 2005.0.1	EW BROWN MAIN PON	D			SUBW= 1
TIME 09:44:56	SITE = EWB1		PASS=	1	PART= 1

\*\*\*\*\*\* CLIMATE AREA - NOT DEFINED

**ENDTABLE** 

WSDATA

ENDJOB

STORM

POOLDATA ELEV

GO, RAINS QSNL

BASIC DATA

\*\*\*\*\*\*\*\*\*\* DESIGN CLASS C

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 28.00	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	CN 95.00	DA-SM 0.15	TC/L 0.25	-/H 0.00	QRF 0.00
SITEDATA-	PERM POOL 924.40	CREST PS 924.40	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 104.40	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND 0.00	COND L 0.00	DIA/W 0.00	-/H 0.00	
	PS N	KE	WEIR L Page 1	TW EL	

	0.000	0.0	EWB_M	ain_P3_PN 0.0	4P.OUT	0.00	
21	D STG 0.00	ORF 0.0	H 00	ORF 0.0	L S	TART AUX. 0.00	
ASCRESTS -	AUX.1 0.00	AUX. 0.0	2	AUX. 0.0	3	AUX.4 0.00	
AUX.DATA - R	EF.NO.	RETARD. 0.0	Ci T	IE STATI	ON INL	ET LENGTH 0	
AUX.DATA - IN	LET N 0.000	SIDE SLO	PE 0	0.00	N EX	IT SLOPE 0.000	ACTUAL AUX
BTM WIDTH -	BW1 0.00	0.0	2	BW:	3	BW4 0.00	BW5 0.00
XEQ 10/18/200 VER 2005.0.1 TIME 09:44:56							WSID= EWB1 SUBW= 1 PART= 2
PERM POOL							CFS
CREST PS	924.40	FT	0.0 AC	CFT 8	8.50 AC	15.4	CFS
SED ACCUM	924.40	FT	0.0 AC	FT 8	8.50 AC	15.4	CFS
BASEFLOW	924.40	FT	0.0 AC	FT 8	8.50 AC	15.4	CFS
RATING TABLE D	DEVELOPED,	SITE = E	WB1:			******	******
RATING TABLE N ELEV. FEET 1 924.40 2 924.50 3 925.00	Q-TOTAL CFS 15.45 17.10	Q-PS CFS 15.45 17.10	Q-	AUX. CFS 0.00 0.00 0.00	VOLUME AC-FT 0.00 8.85	AREA ACRE 88.50 88.55 88.80	

	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	924.40	15.45	15.45	0.00	0.00	88.50
2	924.50	17.10	17.10	0.00	8.85	88.55
3	925.00	26.30	26.30	0.00	53.19	88.80
4	925.50	36.80	36.80	0.00	97.65	89.06
5	926.00	48.30	48.30	0.00	142.25	89.31
6	926.50	60.90	60.90	0.00	186.97	89.57
7	927.00	74.40	74.40	0.00	231.81	89.82
8	927.50	88.80	88.80	0.00	276.79	90.08
9	928.00	1104.00	104.00	1000.00	321.89	90.33
CTODA	LUVD	D= 6.00 L	ID D- 3	28 00 TM	0- 27 38	TN D

STORM HYD D= 6.00 HR P= 28.00 IN Q= 27.38 IN DA= 0.15 SM TC= 0.25 HR CN= 95.00 VOL= 318.2 ACFT

PEAK = 2003.2 CFS, AT 2.5 HRS.

ROUTED RESULT - HYD TYPE EMAX VOL-MAX AMAX QMAX STORM HYD 926.67 FT 202.1 ACFT 89.65 AC 65.4 CFS

PS STORAGE 202.1 ACFT, BETWEEN AUX. CREST AND SED. ACCUM ELEVATIONS.
Page 2

		-		
CMD	Main	D3	PMP	OHT
FWD.	MICE III	F 3	F 1.15	001

AUX.	CREST	927.50		276.8 ACFT	90.08		88.8 CFS ACCUM ELEVATION
BASE	FLOW	924.40		0.0 ACFT	88.50		15.4 CFS
SED A	ACCUM	924.40	FT		88.50		15.4 CFS
CREST	r PS	924.40	FT	0.0 ACFT	88.50	AC	15.4 CFS
PERM	POOL	924.40	FT	0.0 ACFT	88.50	AC	15.4 CFS

PS STORAGE 276.8 ACFT, BETWEEN AUX. CREST AND SED. ACCUM ELEVATIONS.

START ELEV 924.40 FT 0.0 ACFT 88.50 AC 15.4 CFS

STORM HYD D= 6.00 HR P= 28.00 IN Q= 27.38 IN DA= 0.15 SM TC= 0.25 HR CN= 95.00 VOL= 318.2 ACFT

PEAK = 2003.2 CFS, AT 2.5 HRS.

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

RATING TABLE DEVELOPED, SITE = EWB1 : WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RA	TIN	IG TABLE	NUMBER 3				
		ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
		FEET	CFS	CFS	CFS	AC-FT	ACRE
	1	924.40	15.45	15.45	0.00	0.00	88.50
	2	924.50	17.10	17.10	0.00	8.85	88.55
	3	925.00	26.30	26.30	0.00	53.19	88.80
	4	925.50	36.80	36.80	0.00	97.65	89.06
	5	926.00	48.30	48.30	0.00	142.25	89.31
	6	926.50	60.90	60.90	0.00	186.97	89.57
- 5	7	927.00	74.40	74.40	0.00	231.81	89.82
	0	927.50	88.80	88.80	0.00	276.79	90.08
	8	928.00	1104.00	104.00	1000.00	321.89	90.33

# ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 924.40

ROUTED RESULTS	BTM WIDTH	MAX ELEV FT	VOL-MAX ACFT	AREA-MAX AC	FT	VOL-AUX.
STORM HYD	0.0	926.67	202.1	89.7	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

1SITES....JOB NO. 1 COMPLETE.

## EWB1 EW BROWN MAIN POND PHASE 3 PMP

- O SUBWATERSHED(S) ANALYZED.
  - 1 STRUCTURE(S) ANALYZED,
  - 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
  - O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

### SITES....COMPUTATIONS COMPLETE

1	SUMMARY TABLE 1	SITES VERSION 2005.0.1
		DATED 01/01/2005

WATE EWB1	RSHED ID				N DATE 8/2007				RUN TIME 09:44:56
>>>	SITE	SUBWS S	SUBWS DA	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC	<<<
	EWB1	1	0.15	95.	0.25	0.15	TR60	C	
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CREST ELEV (FT)	WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)		ST.	EXIT* VEL. T/SEC)	TYPE HYD
1	0.0	927.5	0.0	-0.8	926.7	0.	0.	0.0 S	TORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

EWB Main\_P4\_100YR.OUT

TIME 09:44:59

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\* 80-80 LIST OF INPUT DATA \*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

SITES	01/01/200	5EWB1	EW BROWN	MAIN POND	PHASE 4	100YR 0.17	33 3
STRUCTURE		EW BROWN	MAIN POND				
		942.40	97.87	15.45			
		942.50	97.92	17.1			
		943.00	98.19	26.3			
		943.50	98.46	36.8			
		944.00	98.72	48.3			
		944.50	98.99	60.9			
		945.00	99.26	74.4			
		945.50	99.53	88.8			
		946.00	99.80	104.0	1000		
ENDTABLE		310.00	33.00				
POOLDATA	ELEV	942.40	942.40				
WSDATA	5C 1 2 AC		110.9	0.25			
	JC I Z AC	33	6	0.25		89.16	
STORM	OCM		4.53			03.10	
GO, RAINS	QSNL		4.03				
ENDJOB							

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\*\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 4.53	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE	
WSDATA -	CN 95.00	DA-SM 0.17	TC/L 0.25	0.00	QRF 0.00	
SITEDATA-	PERM POOL 942.40	CREST PS 942.40	FP SED 0.00	VALLEY FL 0.00	378? NO	
	BASEFLOW 89.16	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING		
PSDATA -	NO. COND 0.00	COND L 0.00	DIA/W 0.00	0.00		
	PS N	KE	WEIR L Page 1	TW EL		

0.000	0.00	Main_P4_100YR.0 0.00	0.00	
2ND STG 0.00	ORF H 0.00	ORF L 0.00	START AUX. 0.00	
ASCRESTS - AUX.1	AUX.2 0.00	AUX.3 0.00	AUX.4 0.00	AUX.5 0.00
	RETARD. Ci 0.00	TIE STATION 0.00	INLET LENGTH	
AUX.DATA - INLET N 0.000	SIDE SLOPE 0.00	EXIT N 0.000	EXIT SLOPE 0.000	ACTUAL AUX?
BTM WIDTH - BW1 0.00	BW2 0.00	BW3 0.00	BW4 0.00	BW5 0.00
VER 2005.0.1		D PHASE 4 100YR WN MAIN POND 1		WSID= EWB1 SUBW= 1 PART= 2
ROUTING OF STORM HYDRO	GRAPH STARTS	AT ELEVATION	942.40	
PERM POOL 942.40	FT 0.0	ACFT 97.87	AC 15.4	CFS
CREST PS 942.40	FT 0.0	ACFT 97.87	AC 15.4	CFS
SED ACCUM 942.40	FT 0.0	ACFT 97.87	AC 15.4	CFS
BASEFLOW 942.40	FT 0.0	ACFT 97.87	AC 15.4	CFS
RATING TABLE DEVELOPED WITH PS AND AUX. GIVE	, SITE = EWB1	:	****	****
RATING TABLE NUMBER 2 ELEV. Q-TOTA FEET CFS 1 942.40 15.45 2 942.50 17.10 3 943.00 26.30 4 943.50 36.80 5 944.00 48.30 6 944.50 60.90 7 945.00 74.40 8 945.50 88.80 9 946.00 1104.00	L Q-PS CFS 15.45 17.10 26.30 36.80 48.30 60.90 74.40 88.80	Q-AUX. VOLUM CFS AC-1 0.00 0.0 0.00 9.7 0.00 58.8 0.00 107.9 0.00 206.7 0.00 256.2 0.00 305.9 1000.00 355.7	FT ACRE 00 97.87 79 97.92 31 98.19 98 98.46 27 98.72 70 98.99 26 99.26 96 99.53	
STORM HYD D= 6.00 TC= 0.25	) HR P= 4. 5 HR CN= 95	53 IN Q= 3 5.00 VOL=	3.95 IN DA 138.7 ACFT	A= 0.17 SM
PEAK = 372.9	CFS, AT 2	.5 HRS.		
COUTED RESULT - HYD TYF		VOL-MAX T 34.8 ACF		QMAX 21.8 CFS

PS STORAGE 34.8 ACFT, BETWEEN AUX. CREST AND SED. ACCUM ELEVATIONS.
Page 2

-		- A	700000	OFFICE
FMR	Marin	DA	100YR.	1.31
LVVD	1.107 1 11		TOOLIN	001

PERM POOL	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
CREST PS	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
SED ACCUM	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
BASEFLOW	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
AUX. CREST	945.50 FT	306.0 ACFT	99.53 AC	88.8 CFS
PS STORAG	SE 306.0 ACF	T, BETWEEN AUX.	CREST AND SEL	O. ACCUM ELEVATIONS.
START ELEV	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
STORM HYD	D= 6.00 HR C= 0.25 HR	P= 4.53 IN CN= 95.00	Q= 3.95 IN VOL= 138.7	DA= 0.17 SM
PFAK =	372.9 CFS.	AT 2.5 HRS.		

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

RATING TABLE DEVELOPED, SITE = EWB1 :

WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RATIN	NG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	942.40	15.45	15.45	0.00	0.00	97.87
2	942.50	17.10	17.10	0.00	9.79	97.92
3	943.00	26.30	26.30	0.00	58.81	98.19
4	943.50	36.80	36.80	0.00	107.98	98.46
5	944.00	48.30	48.30	0.00	157.27	98.72
6	944.50	60.90	60.90	0.00	206.70	98.99
7	945.00	74.40	74.40	0.00	256.26	99.26
8	945.50	88.80	88.80	0.00	305.96	99.53
9	946.00	1104.00	104.00	1000.00	355.79	99.80

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 942.40

ROUTED	BTM WIDTH	MAX ELEV	VOL-MAX	AREA-MAX	AUXHP	VOL-AUX.
RESULTS	FT	FT	ACFT	AC	FT	ACFT
STORM HYD	0.0	942.76	34.8	98.1	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

1SITES....JOB NO. 1 COMPLETE.

EWB1 EW BROWN MAIN POND PHASE 4 100YR 0.1733

O SUBWATERSHED(S) ANALYZED.

1 STRUCTURE(S) ANALYZED.

2 HYDROGRAPHS ROUTED AT LOWEST SITE.

O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

## SITES....COMPUTATIONS COMPLETE

1				SUMMARY	TABLE	1			2005.0.1 01/2005
WATEI	RSHED ID				N DATE 8/2007				RUN TIME 09:44:59
>>>	SITE ID 	SUBWS :	SUBWS DA (SQ MI) 0.17	CURVE NO.	TC (HRS)  0,25	TOTAL DA (SQ MI)	TYPE DESIGN  TR60	STRUC CLASS 	
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CREST		MAX. HP (FT)	MAX. ELEV (FT)	EMB. INT	EGR.* EX	(IT* /EL. /SEC)	TYPE HYD
1	0.0	945.5	0.0	-2.7	942.8	0.	0.	0.0 5	TORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

			DECOURCE	CTTE ANALY	SIS COMPUTER	PROCEAM
VER	2005.0.1 2005.0.1 09:45:03	/ WAIER	(USER MA	NUAL - DAT	ED MAY 2001)	PROGRAM
****	*****	****** 80	)-80 LIST	OF INPUT DA	AIA - AAAA	******
SITES	01/01/200	5EWB1			PHASE 4 PMP	0.1733 3
STRUCTURE	EWB1		MAIN POND			
		942.40		15.45		
		942.50	97.92	17.1		
		943.00	98.19	26.3		
		943.50	98.46	36.8		
		944.00	90.72	48.3 60.9		
			98.99	74.4		
		945.00	99.26	74.4 88.8		
		945.50		104.0	1000	
ENDEADLE.		946.00	99.80	T04.0	1000	
ENDTABLE	EL EVI	042 40	042 40			
POOLDATA	ELEV 5C 1 2 AC	05	110.9	0.25		
	OC I Z AC	93	6	0.23	3	39.16
STORM	OCNI		28			73.10
GO, RAINS ENDJOB	USINL		20			
			-			******
	SAGE - DRAI	NAGE AREA	FROM WSDA	ATA CONTROL	BEING CONVE	RTED FROM
**** MES	SAGE - DRAI AC SAGE - RUN	NAGE AREA	FROM WSDA UARE MILES	ATA CONTROL S FOR COMPL	BEING CONVE	RTED FROM
**** MES **** MES	SAGE - DRAI AC SAGE - RUN IF T	NAGE AREA RES TO SQ OPTION 'S THE FIRST	FROM WSDA UARE MILES ' USED FOR AUX. SPILE	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST	BEING CONVE TATION PURPO , ONLY THE P IS BLANK.	ERTED FROM OSES.
**** MES  **** MES  SITES XEQ 10/18	SAGE - DRAI AC SAGE - RUN IF T	NAGE AREA CRES TO SQ OPTION 'S THE FIRST	FROM WSDA UARE MILES ' USED FOR AUX. SPILE	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST OND PHASE 4	BEING CONVE JTATION PURPO 5, ONLY THE P IS BLANK.	RTED FROM USES. PSH WILL BE ROUTED WSID= EWB1 SUBW= 1
**** MES **** MES	SAGE - DRAI AC SAGE - RUN IF T	NAGE AREA CRES TO SQ OPTION 'S THE FIRST	FROM WSDA UARE MILES ' USED FOR AUX. SPILE	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST OND PHASE 4	BEING CONVE JTATION PURPO 5, ONLY THE P IS BLANK.	ERTED FROM OSES. PSH WILL BE ROUTED WSID= EWB1
**** MES  **** MES  SITES XEQ 10/18 VER 2005.	SAGE - DRAI AC SAGE - RUN IF T	NAGE AREA CRES TO SQ OPTION 'S THE FIRST	FROM WSDA UARE MILES ' USED FOR AUX. SPILE WN MAIN PO EW BROWN M	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST DND PHASE 4	BEING CONVE	WSID= EWB1 SUBW= 1 PART= 1
**** MES  **** MES  SITES XEQ 10/18 VER 2005. TIME 09:4	SAGE - DRAI AC SAGE - RUN IF T	CNAGE AREA CRES TO SQ OPTION 'S THE FIRST EW BROW	FROM WSDA UARE MILES ' USED FOR AUX. SPILE WN MAIN PO EW BROWN M	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST DND PHASE 4	BEING CONVE	RTED FROM DSES. PSH WILL BE ROUTED WSID= EWB1 SUBW= 1 PART= 1
**** MES  **** MES  SITES XEQ 10/18, VER 2005. TIME 09:4	SAGE - DRAI AC SAGE - RUN IF T	NAGE AREA CRES TO SQ OPTION 'S THE FIRST EW BROD	FROM WSDA UARE MILES ' USED FOR AUX. SPILE WN MAIN PO EW BROWN M = EWB1	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST DND PHASE 4	BEING CONVE	RTED FROM DSES. PSH WILL BE ROUTED WSID= EWB1 SUBW= 1 PART= 1
**** MES  **** MES  **** MES  SITES XEQ 10/18, VER 2005. TIME 09:4  ********	SAGE - DRAI AC SAGE - RUN IF T  /2007 0.1 5:03  ************* REA - NOT D	OPTION 'S THE FIRST  EW BROTE  SITE  SEEFINED	FROM WSDA UARE MILES ' USED FOR AUX. SPILI WN MAIN PO EW BROWN M = EWBI BASIC D	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST  DND PHASE 4 MAIN POND  DATA ***	BEING CONVE	RTED FROM OSES. PSH WILL BE ROUTED WSID= EWB1 SUBW= 1 PART= 1 ************************************
**** MES  **** MES  **** MES  SITES XEQ 10/18, VER 2005. TIME 09:4  ********	SAGE - DRAI AC SAGE - RUN IF T  /2007 0.1 5:03  ************* REA - NOT D	OPTION 'S OPTION 'S HE FIRST  EW BROU SITE  SET FOR ALL INFALL DIS	FROM WSDA UARE MILES ' USED FOR AUX. SPILI WN MAIN PO EW BROWN M = EWB1 BASIC D UXILIARY S STRIBUTION	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST  DND PHASE 4 MAIN POND  DATA ***  SPILLWAY IS N (CHAPTER	BEING CONVENTATION PURPO FOR A STATE OF THE PROPERTY OF THE P	RTED FROM USES.  PSH WILL BE ROUTED  WSID= EWB1 SUBW= 1 PART= 1  ***********************************
**** MES  **** MES  SITES XEQ 10/18 VER 2005. TIME 09:4  ******** CLIMATE AIS NRCS DESIG	SAGE - DRAI AC SAGE - RUN IF T  /2007 0.1 5:03  ************ REA - NOT D  TRIBUTION U GN STORM RA	OPTION 'S OPTION 'S HE FIRST  EW BROU SITE  SET FOR ALL INFALL DIS	FROM WSDA UARE MILES ' USED FOR AUX. SPILE WN MAIN PO EW BROWN M = EWB1 BASIC D	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST  DND PHASE 4 MAIN POND  DATA ***  SPILLWAY IS I (CHAPTER	BEING CONVE	RTED FROM USES.  PSH WILL BE ROUTED  WSID= EWB1 SUBW= 1 PART= 1  ***********************************
**** MES  **** MES  SITES XEQ 10/18 VER 2005. TIME 09:4  ******** CLIMATE AIS NRCS DESIG	SAGE - DRAI AC SAGE - RUN IF T  /2007 0.1 5:03  *********** REA - NOT D  TRIBUTION U GN STORM RA	OPTION 'S OPTION 'S HE FIRST EW BROD SITE SEEFINED USED FOR AU INFALL DIS	FROM WSDA UARE MILES ' USED FOR AUX. SPILI WN MAIN PO EW BROWN M = EWB1 BASIC D UXILIARY S STRIBUTION	ATA CONTROL S FOR COMPL R THIS PASS LWAY CREST  DND PHASE 4 MAIN POND  DATA ***  SPILLWAY IS N (CHAPTER	BEING CONVENTATION PURPO FOR A STATE OF THE PROPERTY OF THE P	WSID= EWB1 SUBW= 1 PART= 1  ***********************************

PRECIP	P-LOW 28.00	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	95.00	DA-SM 0.17	TC/L 0.25	-/H 0.00	QRF 0.00
SITEDATA-	PERM POOL 942.40	CREST PS 942.40	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 89.16	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND 0.00	COND L 0.00	DIA/W 0.00	-/H 0.00	
	PS N	KE	WEIR L Page 1	TW EL	

	0.000	0.0	EWB_Ma1	0.00	UT	0.00		
	D STG 0.00	ORF 0.0	H 00	ORF L 0.00		RT AUX.		
ASCRESTS -	AUX.1 0.00		.2	AUX.3 0.00		AUX.4 0.00		AUX.5 0.00
AUX.DATA - R	EF.NO.			E STATION 0.00		LENGTH 0		
AUX.DATA - IN	LET N 0.000	SIDE SLO	PE 00	0.000	EXIT	SLOPE 0.000	ACT	UAL AUX? NO
BTM WIDTH -	BW1 0.00		0	BW3 0.00		BW4 0.00		BW5 0.00
XEQ 10/18/200 VER 2005.0.1 TIME 09:45:03	7	SITE =	BROWN M EWB1	MAIN POND	PASS=		WSID= SUBW= PART=	1
PERM POOL	942.40	FT	0.0 ACF	т 97.87	7 AC	15.4	CFS	
CREST PS	942.40	FT	0.0 ACF	T 97.87	AC.	15.4	CFS	
SED ACCUM	942.40	FT	0.0 ACF	T 97.87	' AC	15.4	CFS	
BASEFLOW	942.40	FT	0.0 ACF	T 97.87	' AC	15.4	CFS	
**************************************	DEVELOPED, NUX. GIVEN NUMBER 2 Q-TOTAL	SITE = EI - NO ASC	EWB1 : DATA REC	ORD GIVEN.	ME	AREA	****	* * * * * * * * * *
1 942.40 2 942.50	15.45 17.10	CFS 15.45 17.10	0		UU	ACRE 97.87 97.92		

17.10 0.00 17.10 9.79 97.92 26.30 26.30 0.00 36.80 36.80 0.00 3 943.00 58.81 98.19 36.80 36.80 0.00 48.30 48.30 0.00 60.90 60.90 0.00 107.98 157.27 206.70 4 943.50 98.46 944.00 5 98.72 6 944.50 98.99 0.00 7 74.40 945.00 74.40 99.26 256.26 88.80 8 945.50 88.80 305.96 0.00 99.53 946.00 1104.00 104.00 1000.00 355.79 99.80 STORM HYD D= 6.00 HR P= 28.00 IN Q= 27.38 IN DA= TC= 0.25 HR CN= 95.00 VOL= 355.1 ACFT

PEAK = 2343.3 CFS, AT 2.5 HRS.

ROUTED RESULT - HYD TYPE EMAX VOL-MAX AMAX QMAX STORM HYD 944.82 FT 237.9 ACFT 99.16 AC 69.4 CFS

PS STORAGE 237.9 ACFT, BETWEEN AUX. CREST AND SED. ACCUM ELEVATIONS.
Page 2

0.17 SM

C1.10	Main	DA	DARCY	OUT
FWR.	Ma In	P4	PIMP	1.11.1

PERM POOL	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
CREST PS	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
SED ACCUM	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
BASEFLOW	942.40 FT	0.0 ACFT	97.87 AC	15.4 CFS
AUX. CREST	945,50 FT	306.0 ACFT	99.53 AC	88.8 CFS
PS STORAGE	306.0 ACFT	, BETWEEN AUX.	CREST AND SEE	O. ACCUM ELEVATIONS.

15 STORAGE SOOTO ACT TO BETWEEN NOW CHEST THE CONTROL OF THE CONTR

START ELEV 942.40 FT 0.0 ACFT 97.87 AC 15.4 CFS

STORM HYD D= 6.00 HR P= 28.00 IN Q= 27.38 IN DA= 0.17 SM TC= 0.25 HR CN= 95.00 VOL= 355.1 ACFT

PEAK = 2343.3 CFS, AT 2.5 HRS.

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RATING TABLE DEVELOPED, SITE = EWB1 :

WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RATII	NG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	942.40	15.45	15.45	0.00	0.00	97.87
2	942.50	17.10	17.10	0.00	9.79	97.92
3	943.00	26.30	26.30	0.00	58.81	98.19
4	943.50	36.80	36.80	0.00	107.98	98.46
5	944.00	48.30	48.30	0.00	157.27	98.72
6	944.50	60.90	60.90	0.00	206.70	98.99
7	945.00	74.40	74.40	0.00	256.26	99.26
8	945.50	88.80	88.80	0.00	305.96	99.53
9	946.00	1104.00	104.00	1000.00	355.79	99.80

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 942.40

ROUTED	BTM WIDTH	MAX ELEV	VOL-MAX	AREA-MAX	AUXHP	VOL-AUX.
RESULTS	FT	FT	ACFT	AC	FT	ACFT
STORM HYD	0.0	944.82	237.9	99.2	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

PEAK - CFS Q-PS Q-AUX. Q-TOT. DISCHARGE = 69.4 0.0 69.4

1SITES....JOB NO. 1 COMPLETE.

EWB1 EW BROWN MAIN POND PHASE 4 PMP

- O SUBWATERSHED(S) ANALYZED.
- 1 STRUCTURE(S) ANALYZED.
- 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
  - O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

## SITES....COMPUTATIONS COMPLETE

1			SUMMARY	TABLE	1			2005.0.1 /01/2005	
WATE	RSHED ID	r.		RU	N DATE				RUN TIME
EWB1				10/1	8/2007				09:45:03
>>>	SITE	SUBWS :	SUBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC CLAS	
	EWB1	1	0.17	95.	0.25	0.17	TR60	С	
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CREST ELEV (FT)	WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)	VOL. DI	ST. \	XIT* VEL. /SEC)	TYPE HYD
1	0.0	945.5	0.0	-0.7	944.8	0.	0.	0.0	STORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

FWB Main P5 100YR.OUT

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SITES	01/01/2005	SEWB1	EW BROWN	MAIN POND	PHASE	5 1	00YR	0.1855	3
STRUCTURE	EWB1	EW BROWN	MAIN POND						
		958.16	106.42	15.45					
		958.50	106.61	22.8					
		959.00	106.88	35.1					
		959.50	107.15	49.0					
		960.00	107.43	64.4					
		960.50	107.71	81.2					
		961.00	107.99	92.8					
		961.50	108.28	101.7					
		962.00	108.56	109.8	1000				
ENDTABLE		302,00	100,30	200					
	ELEV	958.16	958.16						
POOLDATA.	5C 1 2 AC		118.7	0.25					
WSDATA	SC I Z AC	93	6	0.23			83.	30	
STORM	2500		4.53				05.	50	
GO, RAINS	QSNL		4.33						
ENDJOB									

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\*\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

\*\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 4.53	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE	
WSDATA -	CN 95.00	DA-SM 0.19	TC/L 0.25	-/н 0.00	QRF 0.00	
SITEDATA-	PERM POOL 958.16	CREST PS 958.16	FP SED 0.00	VALLEY FL 0.00	378? NO	
	BASEFLOW 83.30	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING		
PSDATA -	NO. COND 0.00	COND L 0.00	DIA/W 0.00	-/H 0.00		
	PS N	KE	WEIR L Page 1	TW EL		

		2 200		B_Main_P	5_100YR.	OUT	0.00	
		0.000	0.00		0.00		0.00	
	2N	0.00	ORF H 0.00		ORF L 0.00	5	TART AUX. 0.00	
ASCR	ESTS -	AUX.1 0.00	AUX.2 0.00		AUX.3 0.00		AUX.4 0.00	AUX.5 0.00
AUX.	DATA - R	EF.NO.	RETARD. Ci 0.00	TIE S	TATION 0.00	INL	ET LENGTH	L
AUX,	DATA - IN		SIDE SLOPE 0.00		XIT N 0.000	EX	T SLOPE 0.000	ACTUAL AUX
втм	WIDTH -	BW1 0.00	BW2 0.00		BW3 0.00		BW4 0.00	
TIME	2005.0.1 09:45:06		SITE = EV	WB1		PASS=	1	SUBW= 1 PART= 2
			FT 0.				.6 15.4	CFS
	ΓPS	958.16					15.4	
SED #	ACCUM	958.16	FT 0.	0 ACFT	106.42	2 AC	15.4	CFS
BASE	-LOW	958.16	FT 0.	0 ACFT	106.42	AC.	15.4	CFS
RATIN	NG TABLE D	EVELOPED,	******** SITE = EWB - NO ASDAT	1:		*****	*****	********
ATIN		Q-TOTAL	Q-PS	Q-AUX.			AREA	
3	958.50 959.00			0.00 0.00 0.00 0.00	0. 36. 89.	00 22 59	ACRE 106.42 106.61 106.88 107.15	

ELEV. Q-TOTAL Q-PS Q-AUX. VOLUME AREA
FEET CFS CFS CFS AC-FT ACRE
1 958.16 15.45 15.45 0.00 0.00 106.42
2 958.50 22.80 22.80 0.00 36.22 106.61
3 959.00 35.10 35.10 0.00 89.59 106.88
4 959.50 49.00 49.00 0.00 143.10 107.15
5 960.00 64.40 64.40 0.00 196.74 107.43
6 960.50 81.20 81.20 0.00 250.53 107.71
7 961.00 92.80 92.80 0.00 304.45 107.99
8 961.50 101.70 101.70 0.00 358.52 108.28
9 962.00 1109.80 109.80 1000.00 412.73 108.56

STORM HYD D= 6.00 HR P= 4.53 IN Q= 3.95 IN DA= 0.19 SM
TC= 0.25 HR CN= 95.00 VOL= 141.2 ACFT

PEAK = 398.1 CFS, AT 2.5 HRS.

ROUTED RESULT - HYD TYPE EMAX VOL-MAX AMAX QMAX STORM HYD 958.51 FT 37.0 ACFT 106.61 AC 23.0 CFS

PS STORAGE 37.0 ACFT, BETWEEN AUX. CREST AND SED. ACCUM ELEVATIONS.
Page 2

EWR.	Main	P5	100YR	OUT
L WILL	346711	_1	TOOLIV	. 001

PERM POOL	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
CREST PS	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
SED ACCUM	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
BASEFLOW	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
AUX. CREST	961.50 FT	358.5 ACFT	108.28	AC	101.7 CFS
PS STORAGE	358.5 ACFT,	BETWEEN AUX	. CREST	AND	SED. ACCUM ELEVATIONS.
START ELEV	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS

PEAK = 398.1 CFS, AT 2.5 HRS.

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RATING TABLE DEVELOPED, SITE = EWB1 : WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RATIN	NG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	958.16	15.45	15.45	0.00	0.00	106.42
2	958.50	22.80	22.80	0.00	36.22	106.61
3	959.00	35.10	35.10	0.00	89.59	106.88
4	959.50	49.00	49.00	0.00	143.10	107.15
5	960.00	64.40	64.40	0.00	196.74	107.43
6	960.50	81.20	81.20	0.00	250.53	107.71
7	961.00	92.80	92.80	0.00	304.45	107.99
8	961.50	101.70	101.70	0.00	358.52	108.28
9	962.00	1109.80	109.80	1000.00	412.73	108.56

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 958.16

ROUTED	BTM WIDTH	MAX ELEV	VOL-MAX	AREA-MAX	AUXHP	VOL-AUX.
RESULTS	FT	FT	ACFT	AC	FT	ACFT
STORM HYD	0.0	958.51	37.0	106.6	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

Q-TOT. Q-PS Q-AUX. PEAK - CFS DISCHARGE = 23.0

1SITES....JOB NO. 1 COMPLETE.

EW BROWN MAIN POND PHASE 5 100YR 0.1855 EWB1

- O SUBWATERSHED(S) ANALYZED.
- 1 STRUCTURE(S) ANALYZED.
- 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
- O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

#### SITES.....COMPUTATIONS COMPLETE

1				SUMMARY	TABLE	1	SITES VI DA	ERSION 2 TED 01/0	
WATE	RSHED ID			RU	N DATE			F	UN TIME
EWB1				10/1	8/2007			C	9:45:06
>>>	SITE	SUBWS S	SUBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC CLASS	<<<
	EWB1	1	0.19	95.	0.25	0.19	TR60	C	
PASS NO.	DIA./ WIDTH (IN/FT)	AUX.CREST ELEV (FT)	BTM. WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)		ST. V	(IT* /EL. /SEC)	TYPE HYD
1	0.0	961.5	0.0	-3.0	958.5	0.	0.	0.0 ST	ORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

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EWB_Main_P5_PMP.OUT
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SITES XEQ 10/18/2007 WATER RESOURCE SITE ANALYSIS COMPUTER PROGRAM VER 2005.0.1 (USER MANUAL - DATED MAY 2001)
TIME 09:45:10

SITES	01/01/200	SEWB1	EW BROWN	MAIN POND	PHASE	5 PMP	0.1855	3	
STRUCTURE		EW BROWN	MAIN POND						
5,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,		958.16	106.42	15.45					
		958.50	106.61	22.8					
		959.00	106.88	35.1					
		959.50	107.15	49.0					
		960.00	107.43	64.4					
		960.50	107.71	81.2					
		961.00	107.99	92.8					
		961.50	108.28	101.7					
		962.00	108.56	109.8	1000				
ENDTABLE		302.00							
POOLDATA	ELEV	958.16	958.16						
WSDATA	5C 1 2 AC		118.7	0.25					
STORM	JC I E AC	33	6			8	3.30		
	OSNL		28						
GO, RAINS ENDJOB	Q3NL		20						
CINDIOD									

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\*\*\*\*\* MESSAGE - DRAINAGE AREA FROM WSDATA CONTROL BEING CONVERTED FROM ACRES TO SQUARE MILES FOR COMPUTATION PURPOSES.

\*\*\*\* MESSAGE - RUN OPTION 'S' USED FOR THIS PASS, ONLY THE PSH WILL BE ROUTED IF THE FIRST AUX. SPILLWAY CREST IS BLANK.

STORM DISTRIBUTION USED FOR AUXILIARY SPILLWAY IS; NRCS DESIGN STORM RAINFALL DISTRIBUTION (CHAPTER 21, NEH4 & TR-60).

PRECIP	P-LOW 28.00	P-HIGH 0.00	P-INCR. 0.00	DURATION 0.00	RF TABLE
WSDATA -	CN 95.00	DA-SM 0.19	TC/L 0.25	-/н 0.00	QRF 0.00
SITEDATA-	PERM POOL 958.16	CREST PS 958.16	FP SED 0.00	VALLEY FL 0.00	378? NO
	BASEFLOW 83.30	INITIAL EL 0.00	EXTRA VOL 0.00	SITE TYPE EXISTING	
PSDATA -	NO. COND 0.00	COND L 0.00	DIA/W 0.00	0.00	
	PS N	KE	WEIR L Page 1	TW EL	

0.0	000		.00		5_PMP.OU 0.00		0.00		
		U	.00						
2ND S			FH		ORF L				
0.	00	.0	.00		0.00		0.00		
ASCRESTS - AUX	.1	AU)	<.2	- /	AUX.3		AUX.4		AUX.5
0.	00	0.	.00		0.00		0.00		0.00
AUX.DATA - REF.	NO.	RETARD.	Ci	TTF S	TATION	TNLET	LENGTH		
AUX.DATA - REF.	0	0.	.00	122	0.00		0		
AUX.DATA - INLET	N	STDF SI	OPF	F)	KIT N	FXTT	SLOPE	ACTU	AL AUX?
0.0		0.	.00	(	0.000	LAL	0.000	7,5010	NO
BTM WIDTH - B	WT.	P	2W7		RW3		BW4		BW5
0.	00	0.	00		0.00		0.00		0.00
KEQ 10/18/2007 /ER 2005.0.1 FIME 09:45:10		SITE	= EWB	1	FOND	PASS=	1	PART=	2
ROUTING OF STORM	HYDRO	GRAPH ST	ARTS	AT ELEV	ATION	958.16			
PERM POOL	958.16	FT	0.0	ACFT	106.42	AC	15.4	CFS	
CREST PS	958.16	FT.	0.0	ACFT	106.42	AC	15.4	CFS	
SED ACCUM	958.16	FT	0.0	ACFT	106.42	AC	15.4	CFS	
BASEFLOW	958.16	FT	0.0	ACFT	106,42	AC	15.4	CFS	
*****	*****	*****	****	*****	****	****	******	******	******
MATING TABLE DEVI					GIVEN.				
RATING TABLE NUME	BER 2								
ELEV. ( FEET 1 958.16	-TOTAL	Q-P	5	Q-AUX.	VOLUM	ME	AREA		
1 958.16	15 A5	15 /	15	0.00	AC-I	00 1	ACKE 06 42		
2 958.50	22.80	22.8	30	0.00	36.2	22 1	06.61		
3 959 00							06 88		

35.10 0.00 49.00 0.00 89.59 959.00 35.10 106.88 3 49.00 143.10 196.74 107.15 4 959.50 49.00 64.40 81.20 5 107.43 960.00 64.40 0.00 81.20 0.00 960.50 250.53 107.71 6 92.80 0.00 7 961.00 92.80 304.45 107.99 8 961.50 101.70 101.70 0.00 358.52 108.28 412.73 962.00 1109.80 109.80 1000.00 108.56 D=  $6.00 \ \text{HR}$  P=  $28.00 \ \text{IN}$  Q=  $27.38 \ \text{IN}$  DA=  $0.19 \ \text{SM}$  TC=  $0.25 \ \text{HR}$  CN= 95.00 VOL=  $372.9 \ \text{ACFT}$ STORM HYD

PEAK = 2507.0 CFS, AT 2,5 HRS.

ROUTED RESULT - HYD TYPE EMAX VOL-MAX AMAX QMAX STORM HYD 960.52 FT 252.3 ACFT 107.72 AC 81.6 CFS

PS STORAGE 252.3 ACFT, BETWEEN AUX. CREST AND SED. ACCUM ELEVATIONS.
Page 2

ITTE-FEE	Main	DIE	DIME	OUT
FWB:	IVIA I I I	P	PWIP.	- 1717 1

PERM POOL	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
CREST PS	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
SED ACCUM	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
BASEFLOW	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS
AUX. CREST	961.50 FT	358.5 ACFT	108.28	AC.	101.7 CFS
PS STORAGE	358.5 ACFT,	BETWEEN AUX.	CREST	AND	SED. ACCUM ELEVATIONS.
START ELEV	958.16 FT	0.0 ACFT	106.42	AC	15.4 CFS

STORM HYD D= 6.00 HR P= 28.00 IN Q= 27.38 IN DA= 0.19 SM TC= 0.25 HR CN= 95.00 VOL= 372.9 ACFT PEAK = 2507.0 CFS, AT 2.5 HRS.

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RATING TABLE DEVELOPED, SITE = EWB1 : WITH PS AND AUX. GIVEN - NO ASDATA RECORD GIVEN.

RATIN	IG TABLE	NUMBER 3				
	ELEV.	Q-TOTAL	Q-PS	Q-AUX.	VOLUME	AREA
	FEET	CFS	CFS	CFS	AC-FT	ACRE
1	958.16	15.45	15.45	0.00	0.00	106.42
2	958.50	22.80	22.80	0.00	36.22	106.61
3	959.00	35.10	35.10	0.00	89.59	106.88
4	959.50	49.00	49.00	0.00	143.10	107.15
5	960.00	64.40	64.40	0.00	196.74	107.43
6	960.50	81.20	81.20	0.00	250.53	107.71
7	961.00	92.80	92.80	0.00	304.45	107.99
8	961.50	101.70	101.70	0.00	358.52	108.28
9	962.00	1109.80	109.80	1000.00	412.73	108.56

ROUTING OF STORM HYDROGRAPH STARTS AT ELEVATION 958.16

ROUTED	BTM WIDTH	MAX ELEV	VOL-MAX	AREA-MAX	AUXHP	VOL-AUX.
RESULTS	FT	FT	ACFT	AC	FT	ACFT
STORM HYD	0.0	960.52	252.3	107.7	0.00	0.0

\*\*\*\* MESSAGE - ROUTING ONLY: NO AUXILIARY SPILLWAY ANALYSIS

PEAK - CFS Q-PS Q-AUX. Q-TOT. DISCHARGE = 81.6 0.0 81.6

1SITES....JOB NO. 1 COMPLETE.

EWB1 EW BROWN MAIN POND PHASE 5 PMP

- O SUBWATERSHED(S) ANALYZED.
- 1 STRUCTURE(S) ANALYZED.
- 2 HYDROGRAPHS ROUTED AT LOWEST SITE.
  - O TRIALS TO OBTAIN BOTTOM WIDTH FOR SPECIFIED STRESS OR VELOCITY.

### SITES....COMPUTATIONS COMPLETE

1			SUMMARY TABLE 1			DATED 01/01/2005		
WATERSHED ID			RU	N DATE				RUN TIME
EWB1			10/1	8/2007			7	09:45:10
>>> SITE ID		JBWS DA (SQ MI)	CURVE NO.	TC (HRS)	TOTAL DA (SQ MI)	TYPE DESIGN	STRUC CLASS	<<<
EWB1	1	0.19	95.	0.25	0.19	TR60	С	
PASS DIA./ NO. WIDTH (IN/FT)	AUX.CREST ELEV (FT)	BTM. WIDTH (FT)	MAX. HP (FT)	MAX. ELEV (FT)		ST. \	(IT# /EL. /SEC)	TYPE HYD
1 0.0	961.5	0.0	-1.0	960.5	0.	0.	0.0 ST	TORM HYD

<sup>\*</sup> INTEGRITY DIST. AND EXIT VEL. VALUES ARE BASED ON THE ROUTED HYDROGRAPH SHOWN UNDER TYPE HYD.

SITES.....SUMMARY TABLE 1 COMPLETED.

NRCS SITES VERSION 2005.0.1,01/01/2005 EWB1 FILES

Appendix B
Selection of Soil Strengths

# Soil Strengths E. W. Brown Generating Station Main Ash Pond Design Mercer County, Kentucky

#### 1. Scope

This section documents the selection of soil strengths for use in the slope stability analyses, for both static and dynamic loading conditions.

# 2. Strengths for Long-Term Static Conditions

The assumed soil properties used in the analysis of stability under long-term (drained) conditions are summarized in Table 1 Except for the gypsum, the properties were taken from previous design work at the site as indicated in the table.

Table 1. Summary of Soil Properties Used in Static Slope Stability Analyses for Long-Term Conditions.

Material	Total Unit Weight (pcf)	c' (psf)	φ' (deg)	Reference
Gypsum	110	0	35	Laboratory tests
Soil 1 Existing Embankment	125	0	30	
Soil 2 Foundation Soil	125	0	32	
Soil 3 Existing Embankment	125	0	28	Ash Pond Modification
Soil 4 Original Embankment	125	0	32	Design (1989)
Soil 5 Rock Fill	115	0	38	Sheets No. 70/71
Soil 6 Fill	118	0	32	and 71/71
Soil 7 Fly Ash	100	0	32	
Zone I	118	0	38	
Zone IIa/24	118	100	28	
Zone IIb/4	118	100	28	Auxiliary Pond
Zone III	118	100	28	Design (2006)
Zone IV	118	0	38	Drawing No.
Zone V	118	0	35	BRO-C-00227
No. 57 Stone	110	0	38	Rev. B
Rubble Zone	118	0	28	
Bedrock	100	Very S	Strong	

Potential failure surfaces assumed to pass above rock surface in the stability analysis.

To support the current design effort, three consolidated undrained triaxial compression tests were performed on compacted samples of gypsum (flue gas desulphurization product) provided by E.ON. The test results indicated an effective stress strength envelope having φ'

of about 43°. However, given the friable nature of gypsum and the lack of field experience with large structures built of this material, a more conservative estimate of strength (c'=0, \( \phi'=35° \)) was used in the analysis.

Note that all of the ash basin deposits were treated as "Soil 7 Fly Ash" in the stability analyses. Mixed fly ash and coarse bottom ash are found in locations throughout the basin (FMSM Engineers 2006). Relatively clean bottom ash, typically having about 10 to 20% fines, has been deposited since 1989 in the north end of basin. However, this deposit does not extend beneath the existing basin embankments, where critical stability conditions exist. In general, it is difficult to distinguish and demarcate zones of "clean bottom ash" in critical areas of the basin. Moreover, in the 1989 design effort, the bottom ash was assumed to have the same strength as the fly ash (c'=0,  $\phi$ '=32°) under long-term, drained conditions. Accordingly, using the same properties ( $\gamma$ =100 pcf, c'=0,  $\phi$ '=32°) for all of the ash basin sediments is a rational simplification for the stability evaluations.

# 3. Strengths for Short-Term Construction Conditions

The new embankments will be constructed over the existing ash deposits, in which excess pore pressures will be generated and then gradually dissipate with time. The rapid generation and dissipation of pore pressures in the ash sediments was measured in the 2005 embankment load tests (FMSM Engineers 2006).

The short-term (during the construction period) stability of the planned embankments can be evaluated using total stress (undrained) strength properties. Total stress strength parameters are needed only for the fly ash, as significant excess pore pressures are not expected in the other basin materials during construction. Materials other than the ash are unsaturated, highly permeable, or sufficiently far from the new construction, such that their shearing resistance can be evaluated using steady-state pore pressures and the effective stress parameters in Table 1.

The undrained (total stress) shear strength properties of the ash deposits were measured with multiple in situ tests during the Field Performance Test (FMSM Engineers 2006). Data from T-bar penetration, handheld vane shear, and borehole vane shear tests are plotted in Figure 1. Estimates of undisturbed strength from the cone penetration test, which are not plotted here, were generally in the range of 100 to 800 psf (residual strengths and sensitivity were not obtained from the CPT).

High values in the strength profiles can be attributed to the presence of coarse material in the immediate vicinity of the test, and should be ignored when selecting undrained strengths for design. Focusing on the lower range of values, most of the undisturbed strengths are in the range of about 150 to 500 psf. Most of the residual strengths are in the range of 100 to 300 psf. Values in the middle of these ranges are recommended for the design evaluations of short-term slope stability: undisturbed S<sub>u</sub>=300 psf and remolded S<sub>u</sub>=200 psf (Table 2). These mid-range values correspond to a sensitivity of 300/200=1.5.

Note that the excess pore water pressures measured under the test embankments in the Field Performance Test were observed to dissipate rapidly (within 5 days or less). Hence, a fully undrained loading condition is probably unlikely to develop in the ash basin during construction, unless an excessively large amount of embankment material is rapidly placed in a small area.

Table 2. Short-term, Undrained Strength Parameters for Fly Ash.

Condition	ф	S <sub>u</sub> (psf)
Undisturbed	0	300
Residual	0	200

# 4. Strengths for Seismic Loadings

Seismic loadings from small-magnitude, local earthquakes and much larger events at greater distances will create critical conditions for the stability of the modified impoundment. In addition to creating lateral inertial loads, earthquake-induced cyclic loading will generate excess pore water pressure in the basin soils; the build-up of pore pressure will weaken the soils and may, in some locations, cause the saturated ash deposits to liquefy.

The selection of design soil strengths, representing the loss of strength due to cyclic pore pressures, is discussed in Sections 4.1 through 4.4. Soil strengths recommended for evaluating seismic stability are summarized in Table 3. Note that effective stress parameters (c' and  $\phi$ ') are specified; this allows for computing the reduced undrained shear strength using the long-term, steady-state pore pressures assumed to exist prior to the seismic event.

Table 3. Summary of Soil Strengths Used in Pseudostatic Slope Stability Analyses for Seismic Load Conditions.

	Above Phre	atic Surface	Below Phre	atic Surface
	c' (psf)	φ' (deg)	c' <sub>red</sub> (psf)	φ' <sub>red</sub> (deg)
Gypsum	0	35	0	29
Soil 1 Existing Embankment	0	30	0	25
Soil 2 Foundation Soil	0	32	0	27
Soil 3 Existing Embankment	0	28	0	23
Soil 4 Original Embankment	0	32	0	27
Soil 5 Rock Fill	0	38	0	38
Soil 6 Fill	0	32	Not applicable*	
Soil 7 Fly Ash	See Table 4		See Table 4	
Zone I	0	38	Not applicable*	
Zone IIa/24	100	28	Not applicable*	
Zone IIb/4	100	28	Not applicable*	
Zone III	100	28	Not applicable*	
Zone IV	0	38	Not applicable*	
Zone V	0	35	Not applicable*	
No. 57 Stone	0	38	Not applicable*	
Rubble Zone	0	28	Not applicable*	
Bedrock	Very S	trong	Very S	trong

No material of this type below the phreatic surface in the cross sections analyzed.

<sup>\*\*</sup> Potential failure surfaces assumed to pass above rock surface in the stability analysis.

Strength parameters for different zones in the ash basin deposits are summarized in Table 4. The selection of undrained strength parameters for liquefied ash is discussed in Section 4.5.

Table 4. Summary of Fly Ash Strengths Used in Pseudostatic Slope Stability Analyses for Seismic Load Conditions

Location in Ash Basin	Assumed Fly Ash Condition	Strength Parameters
Elevations above phreatic surface + 20 ft	Unsaturated No loss in strength	c'=0 φ'=32°
Elevations below	Unliquefied (FS <sub>liq</sub> ≥ 1.4), 20% loss of strength	c' <sub>red</sub> =0 φ' <sub>red</sub> =27°
phreatic surface + 20 ft	Liquefied (FS <sub>liq</sub> ≤ 1.1) Residual strength	S <sub>liq</sub> /o' <sub>vo</sub> =0.04 Min S <sub>liq</sub> =50 psf
Future deposits above the new basin liner	Liquefied Residual strength	S <sub>liq</sub> =50 psf

#### 4.1. Background: Estimation of Dynamic Soil Strength

In their classic work on the seismic deformations of embankment dams, Makdisi and Seed (1977; 1978) recommended using a percentage of the static undrained shear strength to account for the potential loss of shearing resistance in unliquefied soils during dynamic loading. They found that nonliquefiable soils (clayey soils, unsaturated cohesionless soils, and very dense saturated cohesionless soils) exhibit essentially elastic behavior when subjected to many (>100) cycles of stress at 80% or less of their static undrained shear strength. Substantial deformations generally result when these soils are loaded at higher cyclic shear stresses approaching their undrained shear strength. They thus concluded that for unliquefied soils (i.e., soils that exhibit no more than small increases in pore pressure when loaded cyclically), the "cyclic yield strength" is, conservatively, at least 80% of the static undrained strength. Others have since used similar strength reductions to characterize the dynamic yield strength of unliquefied soils, in general.

Hynes-Griffin and Franklin (1984) recommend using reduced, undrained soil strengths in pseudostatic slope stability analyses. Specifically, they recommend using dynamic yield strengths equal to 80% of the undrained strength for both clays and unliquefied, pervious soils. While a reduced consolidated-undrained (R-type) strength envelope should be used for clayey soils, they recommend using a reduced, composite drained-undrained (S-R type) envelope for pervious soils, to avoid relying on the increased shearing resistance due to decreasing pore pressures in dilative granular soils.

Seed and Harder (1990) concluded that soils with a factor of safety against triggering of liquefaction ( $FS_{liq}$ ) greater than 1.4 would experience only minor pore pressure generation under cyclic loading. Moreover, such soils could be represented in stability and deformation analyses using some large fraction of their static shear strength. For their analysis of the Lower San Fernando Dam failure, Seed and Harder (1990) assumed that granular soils with  $FS_{liq} \ge 1.4$  retained 75% of their static strength (determined based on drained conditions) during the earthquake.

In general, there is precedence for assuming that saturated, granular soils will retain at least 80% of their static strength if they do not approach a liquefied state in an earthquake (U. S. Bureau of Reclamation 1999). This condition can be assumed to apply as long as  $FS_{liq}$  is 1.4 or greater. A 20% reduction in strength can be represented using reduced strength parameters,  $c_{red}$  and  $\phi_{red}$ :

$$c_{red} = 0.80 c$$
  
 $\tan \phi_{red} = 0.80 \tan \phi$ 

1

For sandy soils, the pre-earthquake shear strength under drained conditions (computed using effective stress) can be used to estimate the undrained shearing resistance (for total stress analyses) during and immediately after seismic loading. The 20% reduction in strength from the fully drained value is meant to account, in a simplistic manner, for the reduction in the undrained shearing resistance due to the generation of modest pore pressures during dynamic loading for unliquefied sands ( $FS_{liq} \ge 1.4$ ). This approach has the advantage of indexing undrained strengths to  $\phi$ , which can be estimated with relatively good confidence.

#### 4.2. Soils Above the Phreatic Surface

Soils above the assumed phreatic surface (or fly ash more than 20 ft above the phreatic surface, see Section 4.4) are assumed to exhibit no loss in strength due to seismic loading. The strength parameters tabulated in the second and third columns of Table 3 are thus the same as given in Table 1

Excess pore pressures do not readily develop in unsaturated soils, as volumetric strains associated with particle movement are accommodated by the compression of pore air. Unsaturated cohesive soils have increased stiffness due to the matric suction, making the soil more resistant to the development of excess pore pressure when cyclically sheared. In unsaturated, pervious soils, excess pore pressures that may develop in certain locations quickly dissipate into the surrounding soil mass. The gypsum and rock fill, which are critical to the seismic stability of the embankment, have relatively high permeabilities. Accordingly, embankment soils above the assumed phreatic line were assumed to exhibit no strength loss during an earthquake.

#### 4.3. Unliquefied Soils Below Phreatic Surface

Saturated soils below the assumed phreatic surface were assumed to lose 20% of their preearthquake shear strength due to the generation of excess pore water pressures. The parameters in the last two columns of Table 3 were computed accordingly, using Equ. 1.

In the rock fill, the build-up of significant excess pore water pressures will be impeded by the high shearing resistance and permeability of the material. Accordingly, the static shear strength of the rock fill (c'=0,  $\phi$ '=38°) was not reduced for the pseudostatic analyses of seismic loadings.

Shear strengths for the fly ash deposits, which are susceptible to liquefaction, are summarized in Table 4. Where the factor of safety against liquefaction (FS $_{\text{liq}}$ ) is 1.4 or greater in the saturated ash, the shear strength is assumed to be reduced by 20%. These parameters are assumed to apply below and 20 ft above the phreatic surface (see Section 4.4). Residual strengths for the liquefied ash (FS $_{\text{liq}} \le 1.1$ ) are discussed in Section 4.5.

#### 4.4. Ash in the Capillary Fringe

A significant capillary fringe of saturated to nearly saturated material is expected to remain for about 20 ft above the phreatic surface (ground water table) in the ash deposits. During a seismic event, significant excess pore pressures may develop in the capillary fringe within the fly ash. Accordingly, the saturated ash is considered susceptible to liquefaction below the phreatic surface and within the capillary fringe (elevations<phreatic surface+20 ft). Where the FS<sub>lic</sub>≥1.4 in this zone, the strength was reduced 20% (Table 4).

Surface tension (capillarity) causes water to rise in a soil column above the phreatic surface or water table (elevation of zero pore water pressure). The height of capillary rise in a soil varies inversely with the radius the pore spaces, which is related to the grain size distribution. In silts, the maximum height of capillary rise ranges up to about 30 ft (Fredlund and Rahardjo 1993). In the ash basin, the height of ash saturated by capillarity was assumed to be 20 ft.

The existing basin deposits are primarily fly ash, with variable mixtures of coarser bottom ash in some locations. Because of the depositional sequence, thin layers of varying gradation exist in the basin. Coarse layers can act as "capillary breaks", which stop the upward migration of a wetting front due to capillarity. However, because the saturated basin deposits will drain and dewater, the top of the capillary fringe will drop in the basin over time (drying front). Under these conditions, coarser layers in the ash will not prevent the formation of a residual capillary fringe to full height in the fly ash.

More than 20 ft above the phreatic surface, the ash was assumed to be unsaturated and not prone to significant strength loss. The compressibility of the pore air mitigates the development of excess pore pressures and liquefaction in unsaturated, particulate soils. As the saturation of sand is lowered, the cyclic strength increases rapidly (Yang et al. 2004). For example, Ishihara et al. (2001) published data showing that the cyclic strength of one sand roughly doubled as the degree of saturation was reduced from 100% to 90%.

#### 4.5. Liquefied Ash Deposits

The residual undrained strength of the liquefied ash was estimated from CPT data, using the correlation developed by Olson and Stark (2002). The strength ratio  $(S_{iiq}/\sigma'_{vo})$  for the liquefied ash was estimated to be 0.04, with a minimum strength of  $S_{iiq}$ =50 psf (Table 4).

The measured resistance to penetration of the conical CPT tip  $(q_c)$ , in units of stress, is recorded as the cone penetrometer probe is advanced into the soil. Due to details of the probe design, pore water pressures acting behind the cone tip affect the measurement of tip resistance. A corrected cone tip resistance  $(q_t)$  is computed as:

$$q_t = q_c + (1 - A) \cdot u_2$$

Where A is the net area ratio (0.85 for the probe used here) and  $u_2$  is the pore water pressure measured just behind the cone tip. The effect of this correction is more significant in soft, saturated soils where  $q_c$  is small and  $u_2$  is large. While the difference between  $q_c$  and  $q_t$  is generally insignificant in sandy soils, the correction in Equ. 2 is important for characterizing soft, saturated fly ash.

Olson and Stark (2002) normalized the CPT tip resistance with respect to the overburden pressure using:

$$q_{c1} = q_t \frac{1.8}{0.8 + \left(\frac{\sigma'_v}{P_a}\right)}$$

Where  $\sigma'_{v}$  is the vertical effective stress at the location of the measured  $q_{t}$ , and  $P_{a}$  is one atmosphere of pressure in the units of  $\sigma'_{v}$ .

Fourteen CPT explorations were completed in the ash basin in 2005 (FMSM Engineers 2006). The normalized tip resistance values (q<sub>c1</sub>) from each of these tests are plotted in Figure 2. Concentrating on the lower range of the q<sub>c1</sub> data, and ignoring the data peaks, the basin deposits can be characterized by three bands. At the top, the softest deposits are observed above Elev. 870 ft (in CPT-08, CPT-08A, and CPT-9). A slightly stronger stratum is observed below Elev. 870 ft, down to about Elev. 815 ft. The stronger material beneath corresponds to the coarser material deposited prior to 1973.

Liquefaction is possible in the interval approximately between elevations 850 ft and 870 ft in the critical section for stability. This corresponds in elevation to the middle band of material penetrated in the basin in the 2005 CPTs. Focusing on this depth interval, the data in Figure 2 was re-plotted using expanded scales in Figure 3. Over the depth interval between Elevs. 850 and 870 ft, the low-end q<sub>c1</sub> values vary between about 3 and 9 tsf. Accordingly, the ash deposits in this depth interval can be characterized using a value at the lower one-third of this range, or q<sub>c1</sub>=5 tsf=0.5 MPa.

From their analysis of case history data, Olson and Stark (2002) suggested the following relationship:

$$\frac{S_{liq}}{\sigma'_{VQ}} = 0.03 + 0.0143 q_{c1} \pm 0.03$$

Where  $S_{liq}$  is the residual undrained strength of the liquefied soil,  $\sigma'_{vo}$  is the vertical effective stress, and  $q_{c1}$  (MPa) is the normalized CPT tip resistance as defined in Equ. 3. The correlation in Equ. 4 is applicable for values of  $q_{c1} \le 6.5$  MPa.

Based on the lower one-third value of  $q_{c1}$ =0.5 MPa, the residual strength of the liquefied ash is then given by  $S_{liq}/\sigma'_{vo}$ =0.04 (Table 4). A minimum strength of  $S_{liq}$ =50 psf was assumed for low stress regimes ( $\sigma'_{vo}$ <1250 psf); this minimum value corresponds to the lowest field value back-calculated by Seed and Harder (1990).

The cross sections considered in the stability analyses include future ash deposits, which will be will sluiced to the basin above the new liner. These sediments are expected to exhibit properties similar to the existing ash deposits, but will be saturated and under lower effective pressures. Hence, a residual strength of 50 psf was assumed for liquefied ash above the new basin liner (Table 4).

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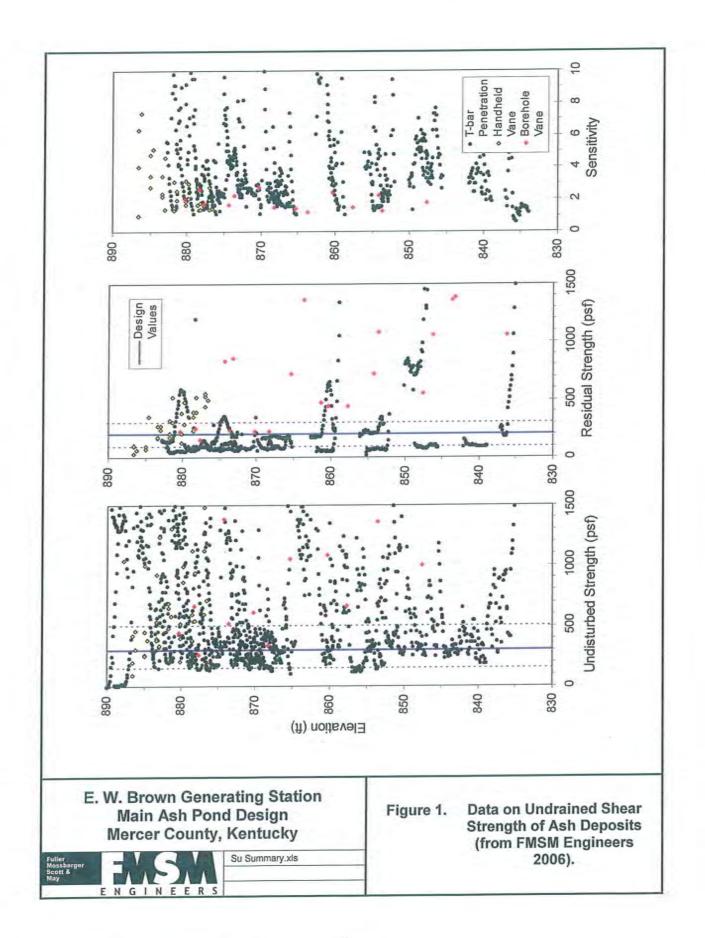
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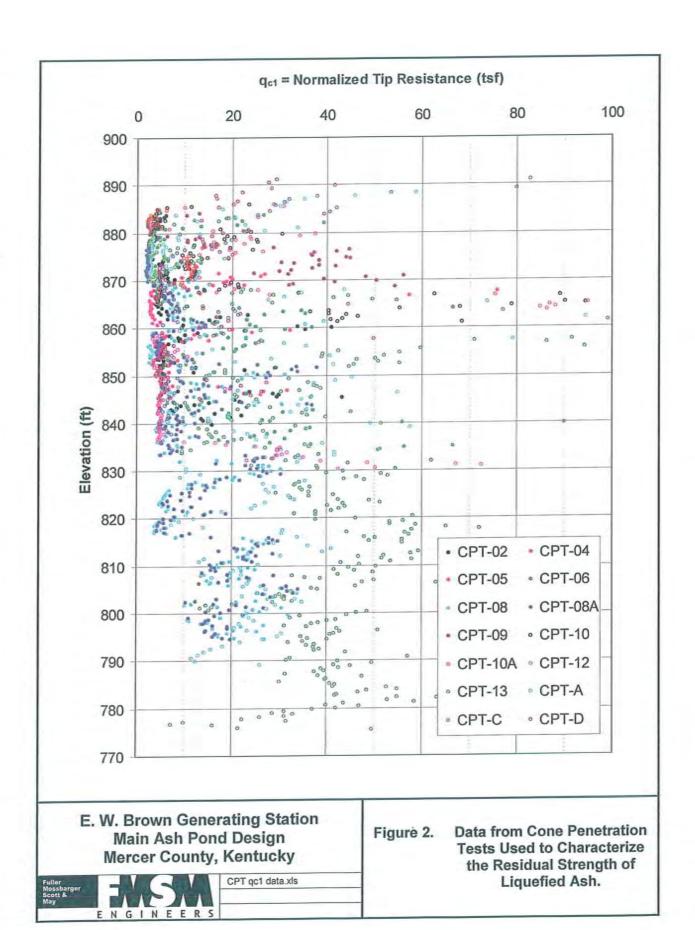
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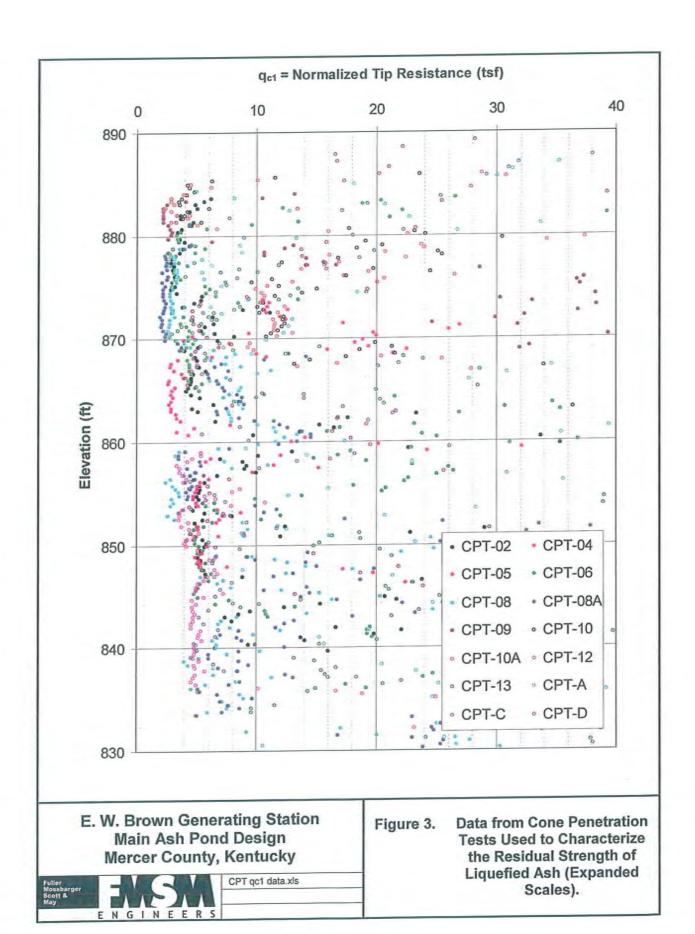
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# Summary of Soil Tests

iect Name	E. W. Brown By	Product Manage	ment Project Number	
	Gypsum		Lab ID	266
			Date Received	2-1-06
	Mercer	_	Date Reported	
mple Type	Bag	_	Date Hepotics	
			Test Results	
Natu	ral Moisture Co	ontent	Atterberg Limits	
	: ASTM D 2216		Test Method: ASTM D 4318 Method	A
	re Content (%):		Prepared: Dry	
			Liquid Limit:	
			Plastic Limit:	Non Plastic
Pai	rticle Size Anal	vsis	Plasticity Index:	
	Method: ASTM		Activity Index:	N/A
	ethod: ASTM D			
	Method: ASTM			
nyurometer	Wieti iou. Ao i w	D 422	Moisture-Density Relation	ship
Dorti	cle Size	%	Test Method: ASTM D 698 Method A	
Sieve Size	1	Passing	Maximum Dry Density (lb/ft³):	92.6
3"	75	1 dooning	Maximum Dry Density (kg/m³):	1483
2"	50		Optimum Moisture Content (%):	
			Over Size Correction %:	N/A
1 1/2"	37.5		OVER GIZE CONSCIENT ISS	
1"	25			
3/4*	19		California Bearing Rati	io
3/8"	9.5		Test Not Performed	
No. 4	4.75	100.0	Bearing Ratio (%):	N/A
No. 10	2	100.0	Compacted Dry Density (lb/ft³):	
No. 40	0.425	99.9	Compacted Moisture Content (%):	
No. 200	0.075	99.4	Compacted Moisture Content (70).	1 444 1
	0.02	11.9		
	0.005	6.7	Specific Gravity	
	0.002	5.6	Test Method: ASTM D 854	
estimated	0.001	5.0	Prepared: Dry	
		1. 1. 0 (0/)	Particle Size:	No. 10
Plus 3 in. ma	terial, not includ	ded: 0 (%)	Specific Gravity at 20° Celsius:	2.36
	*****	LAACUTO	Specific Chavity at 20 Coloido.	
_	ASTM	AASHTO		
Range	(%)	(%)	Classification	
Gravel	0.0	0.0	Unified Group Symbol:	ML
Coarse San		0.1	Group Name:	0.0
Medium Sar		0.5	Group Name.	-
Fine Sand		0.5	-	
Silt	92.7	93.8	AASHTO Classification:	A-4(0)
Clay	6.7	5.6	AASITI O Glassification.	
			× -	



Project	Name
Source	

E. W. Brown ByProduct Management Gypsum Project Number LX2004069 Lab ID 266

Sieve analysis for the Portion Coarser than the No. 10 Sieve

Test Method: ASTM D 422
Prepared using: ASTM D 421

Particle Shape: N/A
Particle Hardness: N/A

Tested By: JWH
Test Date: 02-06-2006

Date Received 02-01-2006

Maximum Particle size: No. 10 Sieve

	%
Sieve Size	Passing
3"	
2"	
1 1/2"	
1"	
3/4"	
3/8"	
No. 4	
No. 10	100.0

Analysis for the portion Finer than the No. 10 Sieve

Analysis Based on: Total Sample

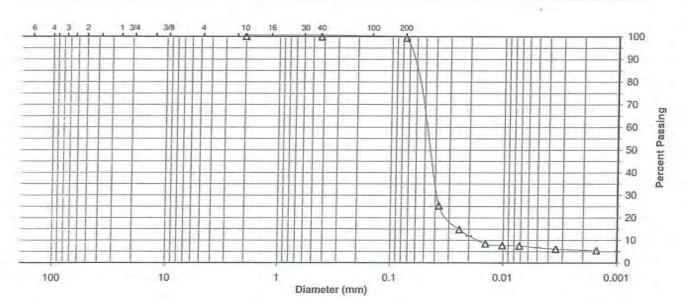
Specific Gravity 2.36

Dispersed using: Apparatus A - Mechanical, for 1 minute

No. 40	99.9
No. 200	99.4
0.02 mm	11.9
0.005 mm	6.7
0.002 mm	5.6
0.001 mm	5.0

#### Particle Size Distribution

ASTM				
AASHTO		-		



Comments

Reviewed By

(22)



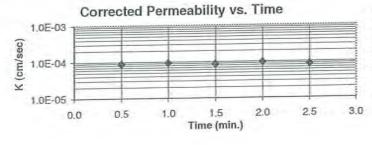
# Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter ASTM D 5084-90

Project No. LX2004069 Project Name E. W. Brown Byproduct Management Test ID 266@248 Source Gypsum Prepared By KDG Silt (ML), tan Visual Classification 2-15-06 Date ASTM D854-A Specific Gravity Compacted Std. 0 in. spacer 100.3 Percent of Maximum 92.6 Maximum Dry Density (pcf) De-aired tap water Permeant: Standard Effort, -No.4 material. Selection and Preparation Comments:

Specimens (if compacted) were compacted in a Proctor Mold as follows: The Maximum Dry Density was converted to Wet Density, this mass was divided by 4 (layers) and 3 of the 4 layers were compacted into the mold using a Proctor Hammer using 19 blows per layer. The density was varied by reducing the height of the drop by the amount listed beside "Compacted". The specimen was trimmed from the bottom two layers.

	Initial Specimen Data	After Consolidation Data	After Test Data	Final Pressures	(psi)	
Height (in.)	1.3970	1.3817	1,3833	Chamber	75	
Diameter (in.)	2.8030		2.7753	Influent	70	
Moisture Content (%)	18.1		23.7	Effluent	65 Applied Head Difference (psi)	5
Dry Unit Weight (pcf)	92.9		95.7		Back Pressure Saturated to (psi)	65
Void Ratio	0.586		0.539	Maximu	m Effective Consolidation Stress (psi)	10
Degree of Saturation (%)	73.1		103.5	Minimu	m Effective Consolidation Stress (psi)	5

							Hydraulic C	Conductivity	
Date	Clock (24H:M)	Temp. °F	Bottom Head	Top Head	Test Time (sec)	k (m/s)	k (cm/s)	k @ 20° C (m/s)	k @ 20° C (cm/s)
2-16-06	10:21	76,0	20.86	4.60	0			***	-
2-16-06	10:21	76.0	17.32	8,06	3.00E+01	9.8E-07	9.8E-05	8.9E-07	8.9E-0
2-16-06	10:22	76.0	13.76	11,56	3,00E+01	1.0E-06	1.0E-04	9.4E-07	9.4E-0
	10:22	76.0	10.67	14.62	3.00E+01	9.6E-07	9.6E-05	8.6E-07	8,6E-0
2-16-06	10:23	76.0	7,35	17.82	3.00E+01	1.1E-06	1.1E-04	9.7E-07	9.7E-0
2-16-06		76.0	4,59	20.64	3.00E+01	9.7E-07	9.7E-05	8.7E-07	8.7E-0
2-16-06	10:23	76.0	4.09	20.04	O,UULTUT			i i	
						1			
-									
								10	
-									



A gradient of approximately 98.8 was used for this test. This gradient exceeds ASTM guidelines for maximum gradient, but was used to achieve the requestors desired test duration. Examination of the sample shows no signs of material loss or clogging that may affect test results.

Average Hydraulic Conductivity @ 20° C (last 4 determinations) Average Hydraulic Conductivity @ 20° C (last run) m/s 9.12E-07 m/s 9.07E-07 cm/s 9,12E-05 cm/s 9,07E-05

Reviewed by:



# Moisture-Density Data Sheet

Project: E. W. Brown ByProduct Management

Source: Gypsum

Prepared: Moist

Sample Description: Silt (ML), tan

Visual Notes:

Oversized Fraction: 0 % Rammer: Manual

Project No.: LX2004069

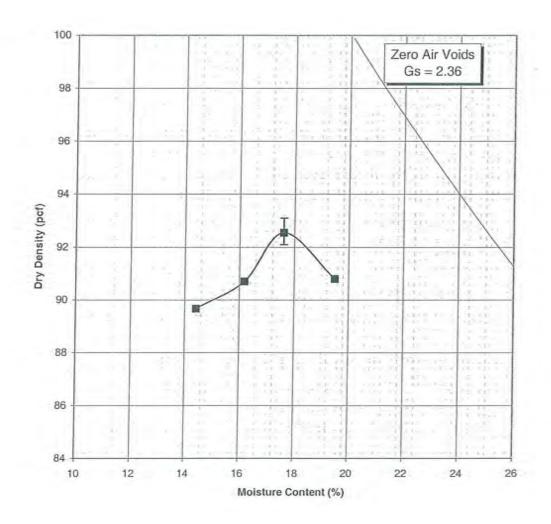
Sample No.: 266

Nmc: 10.6 %

Test Method: ASTM D 698 - Method A

Gs - Fines: ASTM D 854

Mold Weight	4341 grams					
Wet Weight plus Mold (grams)	Wet Weight minus Mold (grams)	Wet Soil and Can Weight (grams)	Dry Soil and Can Weight (grams)		Water Content (%)	Dry Density (pcf)
5886	1545	601.90	535.46	74.59	14.4	89.7
5928	1587	651.29	571.00	74.96	16.2	90.7
5980	1639	639.65	555.00	74.08	17.6	92.5
5975	1634	649.40	555.90	76.44	19.5	90.8



Maximum Dry Density 92.6 PCF Optimum Moisture Content 17.6 %



#### Minimum Index Density ASTM D 4254

Project Name	E.W. Brown Ash Pond Extension Project	
Source	Gypsum	
Description	Silt (ML), tan	

Project Number Lx2004069 Lab ID

Max. Particle Used In Test No. 4

Mold Size

Tare ID Mold Mass (g) 3702.1 0.5 Cubic Foot 0.1 Cubic Foot

Mass of Mold and soil (grams)	Test 1 6390.1	Test 2 6357.8	Test 3
Soil Mass (grams)	2688	2655.7	-3702.1
Minimum Index Density (pcf)	59.4	58.7	

59.1 Average Minimum Index Density (pcf) \_\_\_



#### Maximum Index Density ASTM D 4253

Project Name E.W. Brown Ash Pond Extension Project

Source Gypsum

Description Silt (ML), tan

Project Number Lx2004069 Lab ID

Test No

Mold Size Max. Particle Used In Test No. 4

Vibratory Table Setting 55@8 min

0.5 Cubic Foot 0.1 Cubic Foot

Initial dial Readings (in) (Top of Mold)

0.982 0.982 0.982 0.982 0.982 0.982 0.981 0.981 0.979 0.980 0.981 0.982

Final Dial Readings (in) (Top of Surcharge Plate)

_	(TOP OF SUICE	naige i late)
	2.041	2.074
	2.046	2.077
L	2.052	2.085
	2.044	2.087
	2.052	2.089
	2.056	2.084

0.981 (RI) Average Average 2.065 (RF)

Tare ID A

Tare and Dry Soil (grams) 4025.2 Tare Weight (grams) 1988 Sample Mass (grams) 2037.2

Maximum Index Density (pcf)



#### Maximum Index Density ASTM D 4253

Project Name E.W. Brown Ash Pond Extension Project

Source Gypsum

Description Silt (ML), tan

Project Number <u>Lx2004069</u>

Lab ID <u>266</u>

Test No 2

Mold Size

0.5 Cubic Foot

0.1 Cubic Foot X

Max. Particle Used In Test No. 4
Vibratory Table Setting 55 @ 8 min

Initial dial Readings (in)
(Top of Mold)

	(10p or in	oluj.
	0.985	0.985
	0.986	0.985
	0.985	0.984
7	0.984	0.984
	0.984	0.981
	0.985	0.985

Final Dial Readings (in)
(Top of Surcharge Plate)

(10	p or Surcha	ye riale
	2.668	2.728
	2.674	2.740
	2.677	2.739
	2.683	2.735
	2.697	2.742
	2.694	2.751

Average 0.984 (RI)

Average

2.711 (RF)

Tare ID	1147
Tare and Dry Soil (grams)	3520.2
Tare Weight (grams)	1778.3
Sample Mass (grams)	1741.9

Maximum Index Density (pcf) 81.1

Laboratory Document Prepared By: JW Approved By: TLK





Project Name E.W. Brown Ash Pond Extension Project

Source Gypsum

Description Silt (ML), tan

Project Number Lx2004069 Lab ID Test No

Mold Size

0.5 Cubic Foot

0.1 Cubic Foot

Max. Particle Used In Test No. 4 Vibratory Table Setting 55 @ 8 min

> Initial dial Readings (in) (Top of Mold)

110p of 11	( Top of Mola)	
0.983	0.977	
0.982	0.977	
0.983	0.977	
0.982	0.977	
0.980	0.975	
0.982	0.978	

Final Dial Readings (in)

-	Top of Surc	harge Plate)
	3.211	3.244
	3.233	3.225
	3.227	3.216
	3.232	3.229
	3.238	3.227
	3.236	3.238

Average 0.979 (RI) Average 3.229 (RF)

Tare ID BD-2

Tare and Dry Soil (grams) 3149.1 Tare Weight (grams) 1700.5 Sample Mass (grams) 1448.6

Maximum Index Density (pcf)



#### Consolidated Undrained Triaxial Test ASTM D4767-95

Project Sample ID E.W. Brown Byproduct Management

Project No. Test Number

LX2004069

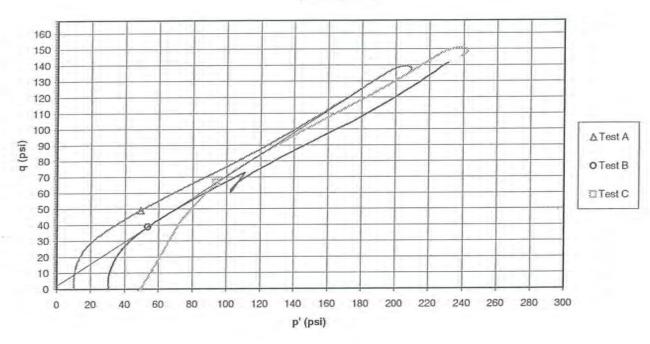
 $\phi' = 43.1 \text{ deg.}$ 

c' = 390 psf

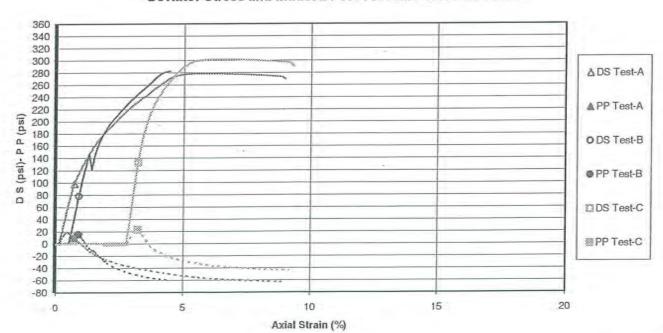
Failure Criterion:

Maximum Effective Principal Stress Ratio

#### p' vs. q Plot



#### Deviator Stress and Induced Pore Pressure vs. Axial Strain



KDG

Laboratory Document Prepared By: MW Approved By: TLK

Appendix C

Selection of Seismic Design Parameters

# Selection of Seismic Design Parameters E. W. Brown Generating Station Main Ash Pond Design Mercer County, Kentucky

#### 1. Scope

This section documents the selection of design earthquake parameters (magnitude and peak ground motion) for use in evaluating the stability of the main ash pond embankments. Parameters were estimated using data from the National Seismic Hazard Mapping Project-a site-specific seismic study was not performed.

### 2. Seismicity

Each seismic source, such as a fault or a zone of known seismicity, can produce earthquakes of different magnitudes at various recurrence intervals (or return periods). The relationships between earthquake size, proximity, and frequency of occurrence should be considered in selecting seismic parameters for design.

Kentucky regulations require the consideration and evaluation of potential earthquake loadings, but do not specify how seismic parameters should be selected. Given the potential consequences of failure, the embankments containing the ash basin at the E.W. Brown facility should be designed and built to survive significant earthquake events, which might occur in that geographic setting during the design life of the project. Consistent with general practice for facilities of this type, the design earthquake parameters were selected to represent a seismic event having a 2% chance of exceedance during a 50-year period.

Earthquake hazards are usually quantified using a Poisson model, wherein the probability of exceedance (PE) is given by:

$$PE = 1 - e^{-\lambda t}$$

Where  $\lambda$  is the annual rate of an event occurring and t is the time in years. The return period (recurrence interval) is the inverse of  $\lambda$ . Assuming t=50 years and PE=0.02,  $\lambda$  is then 0.000404 events per year, which corresponds to return period of 2,475 years.

#### 3. Source of Data

Design earthquake parameters were obtained from United States Geological Survey National Seismic Hazard Maps, available at <a href="http://earthquake.usgs.gov/research/hazmaps">http://earthquake.usgs.gov/research/hazmaps</a>. The National Seismic Hazard Mapping Project produces national maps showing earthquake ground motions for a variety of return periods. The 2002 dataset (revised in 2003) for the 48 conterminous United States was utilized through the "interactive deaggregation" feature. Probabilistic analyses are performed to examine ground motions that could be expected at a site for various recurrence intervals. The motions assume the site is on rock having an

average shear wave velocity of 760 meters per second in the upper 30 meters of material. The map data indicates the overall or total hazard at a site, and is not specific to single potential earthquake sources. A full description of the hazard model and its underlying assumptions is available at the USGS website.

Using the USGS website, ground motion parameters can be obtained by entering the coordinates for a particular site. The latitude and longitude for the E.W. Brown site were estimated using USGS topographic mapping (<a href="http://www.topozone.com">http://www.topozone.com</a>). The approximate coordinates in the middle of the ash treatment basin are:

- Latitude=37.7880 degrees North (NAD83)
- Longitude=84.7191 degrees West (NAD83)

More exact coordinates are not required, as the seismic hazard data are only available on grid points of approximately 0.05 degrees (latitude and longitude). The ash basin is small enough such that a single set of coordinates adequately represents the site.

### 4. Probabilistic Seismic Hazard Analysis

The USGS web site provides tools to estimate the seismic hazard for a specified location and return period. A probabilistic seismic hazard analysis (PSHA) is performed to estimate the total hazard contributions from many different earthquake sources of varying magnitude and varying distance from the site. Top-of-rock peak ground accelerations (PGA<sub>rock</sub>) at the E.W. Brown site, obtained from the PSHA for three different recurrence intervals, are summarized in Table 1. Events with a 2,475-year return period were examined in greater detail to select design ground motion parameters.

In addition to PGA, engineering analyses require other strong motion parameters (earthquake magnitude, etc.) and predicted ground motions (acceleration time histories). The USGS web site provides a "deaggregation" tool to aid the user in coupling an acceleration time history with magnitude and epicentral distance. The total hazard is broken down into combinations of magnitude and distance that are capable of producing the desired event. The deaggregation can be presented geographically to help visualize seismic source zones that are contributing significantly to the total hazard. The deaggregation can also be presented in terms of a magnitude-epicentral distance histogram. The geographic and histogram deaggregations of the seismic hazard for the 2,475-year design event at the E.W. Brown site are shown in Figure 1 and Figure 2, respectively.

The deaggregation tool also returned the modal and mean epicentral distance and magnitude pairs in Table 2 for 2,475-year events at the project site. The modal and mean (R,M) pairs help to characterize the conditional distribution of source properties for a given probability of exceedance at the site (USGS 2006):

- The modal (R,M) pair corresponds to an earthquake that is more likely than any other pair to produce a given ground-motion exceedance in the future. The mode tends to be very likely for sites near short recurrence time source zones, such as the New Madrid Seismic Zone.
- The mean (R,M) pair is an average over all sources considered in the PSHA. The mean (R,M) event is less likely to occur than the mode, for a given PE.

For the E.W. Brown site, the mean pair corresponds to a seismic event about 150 km distant and not associated with a known fault, while the modal event corresponds to an earthquake occurring in the New Madrid Seismic Zone. The contribution of the modal event to the seismic hazard at the site is evident in the graphical presentations in Figure 1 and Figure 2.

While the modal event is of larger magnitude, attenuation of the expected ground motions over the greater epicentral distance results in lower (compared to the mean event) predicted peak accelerations at the project site. The USGS website provides stochastically generated acceleration time histories for the modal and mean events. The SMSIM\_TD program (version 2.20), created by David Boore of USGS, is used to generate synthetic accelerograms for the modal or mean distance/magnitude pairs. The accelerograms are ranked according to how well the spectral acceleration values fit the uniform hazard spectrum, and the six motions with the best fit (A1 through A6) are presented to the user. The six ground motions predicted for the E.W. Brown site during the modal and mean, 2,475-year events are presented in Figure 3 and Figure 4, respectively. The median PGA from the six accelerograms is indicated at the top of each plot, and tabulated in Table 2.

For a 2,475-year return period, the total seismic hazard indicates PGA=0.098 g at the site (Table 1), which is higher than the PGA values predicted for the mean or modal earthquake events (Table 2). From Figure 1 and Figure 2, it appears that the maximum PGA of 0.098 g is predicted for a local event within 50 km of the site. Based on the deaggregation data, it appears that a PGA of 0.098 g is associated with an assumed local event having a magnitude of about  $5.0~\rm M_w$ .

Table 1. Probabilistic Ground Motion Parameters for the E.W. Brown Project Site (USGS 2006)

Probability of Exceedance in 50 years	Return Period (years)	PGA <sub>rock</sub> (g)
2%	2,475	0.098
5%	975	0.059
10%	475	0.039

Table 2. Modal and Mean 2,475-year Events from the PSHA for the E.W. Brown Project Site (USGS 2006)

	Modal Event	Mean Event
Source Zone	New Madrid	Central/Eastern US
Epicentral Distance, R (km)	392	146
Magnitude (M <sub>w</sub> )	7.70	6.32
Median PGA (g)	0.037	0.053

# 5. Recommended Design Earthquake Parameters

Assuming a 2,475-year design event, the PSHA results suggest three earthquake scenarios (Table 3) for the E.W. Brown site. Dynamic slope stability should be evaluated using the local event, with maximum PGA=0.098 g

In evaluating the potential for liquefaction in the basin deposits, the modal event (New Madrid) should be used. As described in more detail later, the factor of safety against liquefaction (Youd et al. 2001) is defined as:

$$FS_{liq} = \frac{CRR}{CSR}$$
 2

The cyclic stress ratio (CSR) represents the dynamic, earthquake loading on the soil. When estimated using the simplified procedure, CSR is linearly proportional to PGA. The cyclic strength of the soil is represented by the cyclic resistance ratio (CRR). Because larger magnitude earthquakes induce more cycles of dynamic loading, CRR (computed using magnitude scaling factors) is inversely related to magnitude. Using the recommended magnitude scaling factors, CRR is inversely proportional to  $M_w^{2.56}$ . When taken together, minimum values of FS<sub>liq</sub> will be computed for maximum values of (PGA· $M_w^{2.56}$ ). This parameter, tabulated in the last column of Table 3 for these three scenarios, indicates that the critical seismic event for the evaluation of soil liquefaction at this site is the modal event (PGA=0.037and Mw=7.70).

Note that the accelerations listed in Table 1 and Table 3 represent top-of-rock motions. Depending the analysis, these PGA<sub>rock</sub> values may need to be modified to account for local site effects. Ground response analysis can be used to model the attenuation and/or amplification of accelerations as the seismic waves travel through the soil column. In general, ground response analyses are warranted where the structure or failure mechanism is not founded on rock.

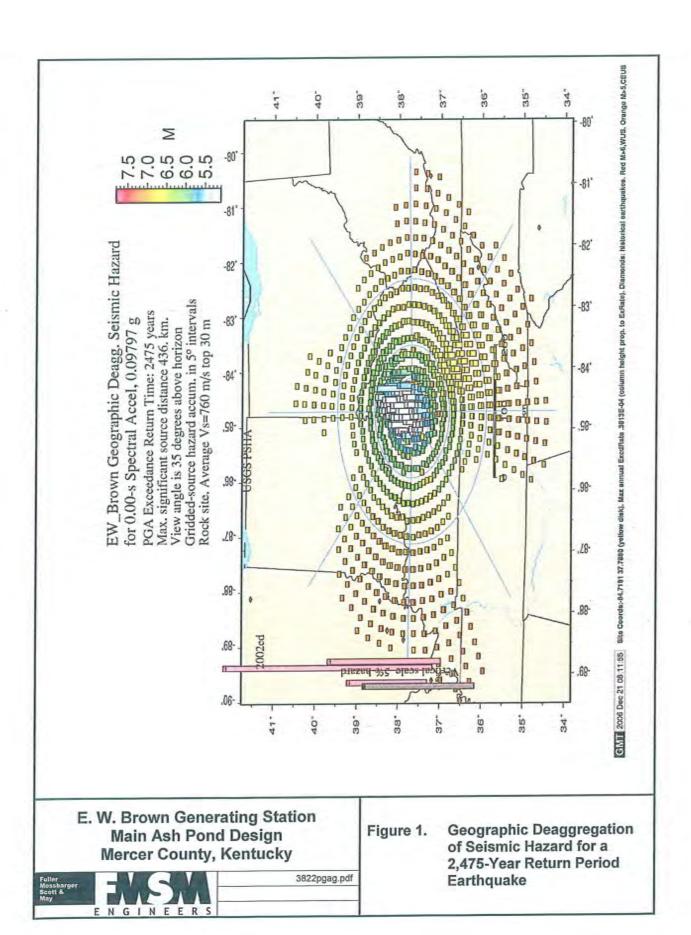
Table 3. Design Earthquake Scenarios for the E.W. Brown Site Assuming a 2% Probability of Exceedance in 50 Years (Return Period Of 2,475 Years).

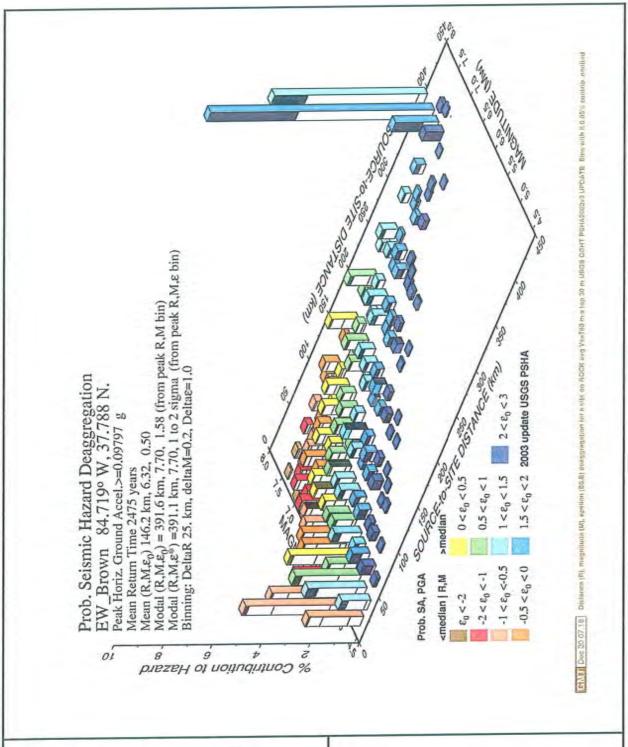
Scenario	Earthquake Magnitude (M <sub>w</sub> )	Top of Rock Peak Ground Acceleration (g)	PGA·M <sub>w</sub> <sup>2.56</sup>
Local Event	5.00	0.098	6.03
Mean Event	6.32	0.053	5.94
Modal Event	7.70	0.037	6.88

#### References

United States Geological Survey (2006). National Seismic Hazard Mapping Project web site, http://eqhazmaps.usgs.gov/.

Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. E., Jr., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson . W. F., III, Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H., II. (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils," *J. Geotechnical & Geoenvironmental Eng., ASCE* Vol. 127, No. 10, pp. 817-833.





E. W. Brown Generating Station Main Ash Pond Design Mercer County, Kentucky

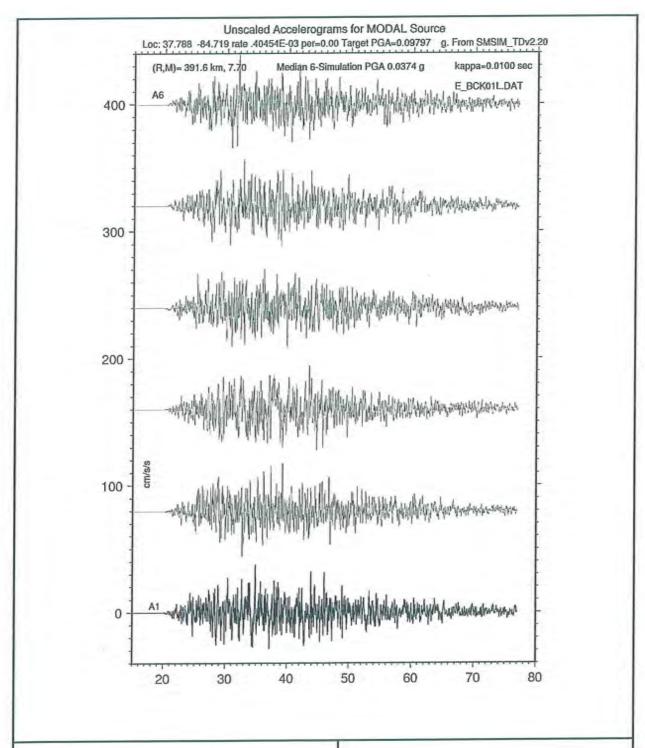
Fuller Mossbarger Scott & May



ntucky

15210pga.pdf

Figure 2. Histogram Deaggregation of Seismic Hazard for a 2,475-Year Return Period Earthquake



E. W. Brown Generating Station Main Ash Pond Design Mercer County, Kentucky

Mossbarger Scott & May

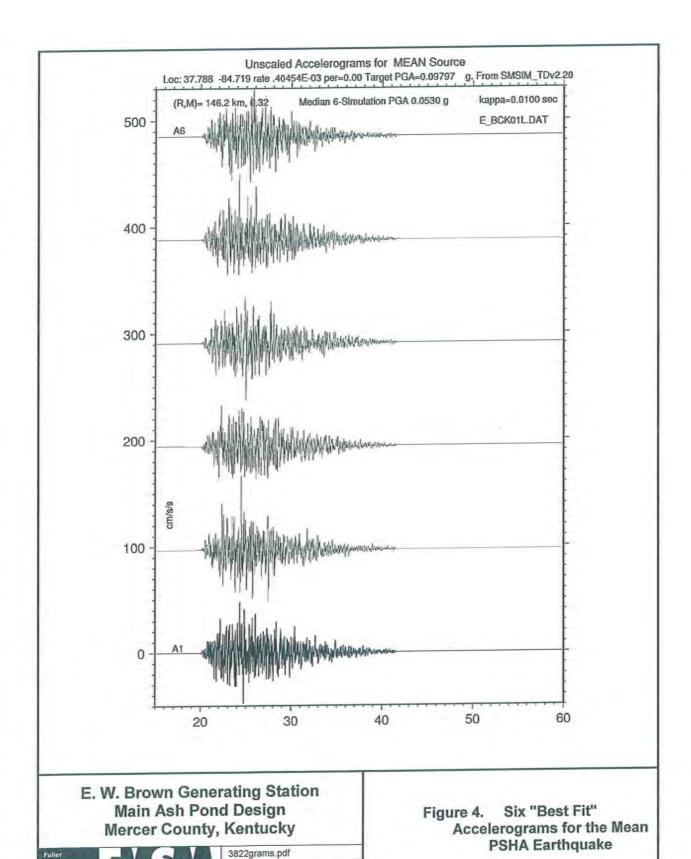


12750grams.pdf

Figure 3. Six "Best Fit"

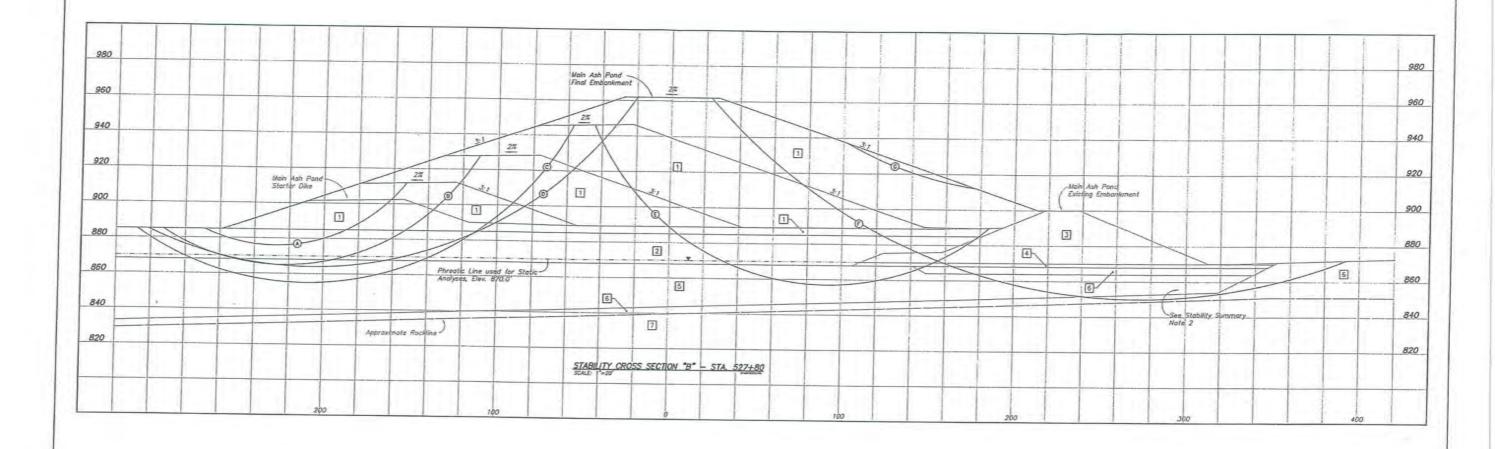
Accelerograms for the

Modal PSHA Earthquake



ENGINEERS

Appendix D
UTEXAS4 Output



_	SUMMARY OF SLOPE	SIMDILITY A	INAL ISES	
Failure Surface	Failure Condition	Failure Mode	Factor of Safet	
	Long Term, Upstream	Rotational	Static	2.7
	Phase 2 - Na Paal	Hotorional	Dynamic <sup>(1)</sup>	23
8	Long Term, Upstream Phase 3 - No Pool	2111	Static	26
		Rotational	Dynamic <sup>(1)</sup>	2.1
c	Lang Term, Upstream Phase 4 - No Pool Rotational	Static	2.5	
		notational	Dynamic(1)	1.9
0	Long Term, Upstream		Static	2.3
-	Phase 5 - No Pool	Rotational	Dynamic(1)	1.8
E	Long Term, Downstream	Rotational	Stotie	2.8
-	Phose 4	Motorional.	Dynamic (1)	2.1
F	Long Term, Downstream	Rotational	Static	2.2
	Phase 5 Deep Failure Hotati	MATGRANA	Dynamic (1)	1.6
G	G Long Term, Downstream Phase 5 Shallow Failure R	Datation of	Stotic	21
		Retational	Dynamic(1)	1.3

The factors of safety under dynamic (pseudo-static) loading conditions are based on a psick ground acceleration (kmm) value of 0.100g for a 2 percent probability of exceedance in 50 years.
 Ultimate Long Term Phreatic Surface must be below Elex 856' to avoid potential liquefaction of the underlying fly ash.

Moterial No.	Description	Effective Stress		
		C (p.s.f.)	\$ (dea.)	y (p.c.t.,
1 2 3	Gjpsum	0	35	110
	Fly Ash	0	32	100
	Existing Embankment	0	32	118
4	Old Warking Plotform	0	35	115
6	Bottom Ash	0	32	115
	Foundation Soil	0	32	100
7	Bedrock	0	Very Strang	125

NOTE: Refer to Design Report for discussion of material zones and material strength parameters.

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL					
COMPRESSION TEST RESULTS					
Soil No.	ø (deg.)	C (p.s.t.)			
1	43.1	390			

SUMMARY OF DIRECT SHEAR TEST RESULTS
Soil No. 5 (deg.) | C (p.s.f.) 2 31.6 72

ENGINEERS

STABILITY ANALYSES
SECTION B - STA. 527+80
MAIN ASH POND - STARTER DIKE

Dorston and Delta E.M. BROWN GENERATING STATION

Kentucky Utilities Company

Code: 3" - 30"

Design: \$13

Design: \$13

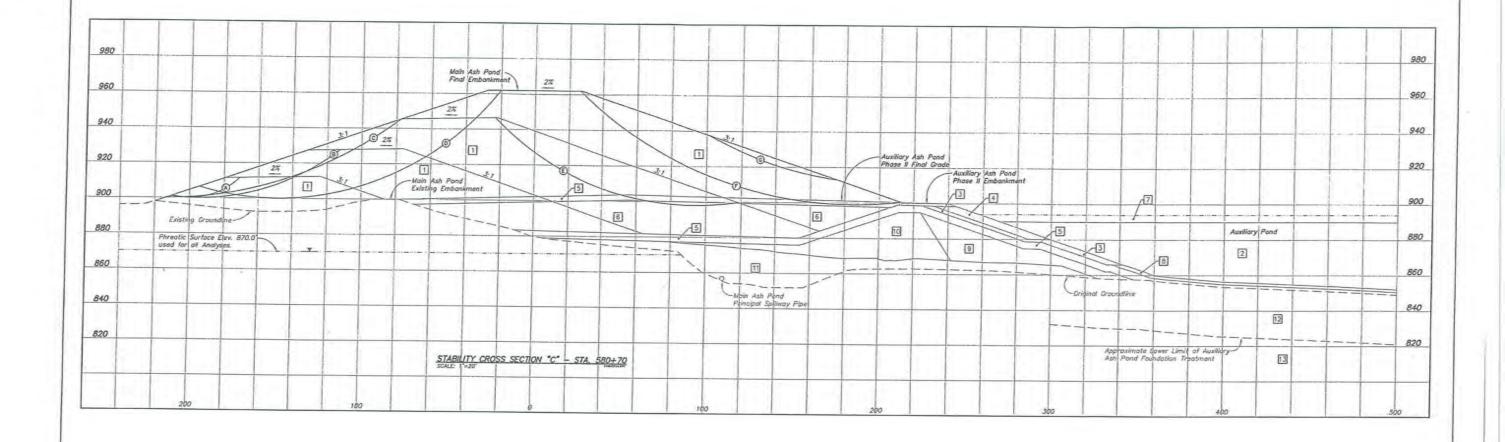
Design: \$13

Design: \$15

De BR0-C-01000

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	SUMMARY OF SLOPE	STABILITY A	INAL TSES	
Failure Surface	Failure Candition	Failure Mode	Factor of Safety	
A	Long Term, Upstream Phase 2 - Na Paol	Rotational	Static	2.6
			Dynamic(1)	2.2
В	Long Term, Upstream Phase 3 — No Pool	Rotational	Static	2.5
			Dynamic <sup>(1)</sup>	2.1
C	Long Term, Upstream Phase 4 — No Pool	Relational	Stotic	2.2
			Dynamic(1)	1.9
D	Long Term, Upstream Phase 5 - No Pool	Rotational	Static	2.5
			Dynamic(1)	1.9
E	Long Term, Downstream Phase 4	Rotational	Static	2.4
			Dynamic(1)	20
F	Long Term, Downstream Phase 5 Deep Fallure	Rotational	Static	2.2
			Dynamic(1)	1.9
G	Long Term, Downstream Phase 5 Shallow Failure	Rotational	Static	2.1
			Dynamic(1)	1.3

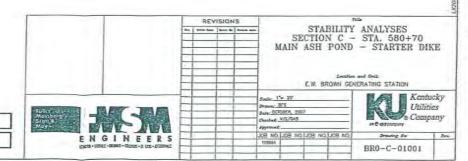
G	Lang Term, Downstream	Rotational	Static	2.1
	Phase 5 Shallow Failure	3000 DOOR DE	Dynamic(1)	1.3
(t) Th	e factors of safety under dynan e based on a peak ground acce	nic (pseudo-st	atic) looding of	anditions

Material	Description	Effective Stress			
No.		C (p.s.t.)	Ø (dea.)	7 (p.c.t.)	
1234567	Gypsum	0	35	710	
	Fly Ash	0	32	100	
[3]	No. 57 Stone	0	38	110	
4	Rock Fill	0	38	118	
5	Clay	100	28	118	
6	Gypsum	0	35	118	
7	Water	0	0	52.4	
8	Bottom Ash	0	38	118	
9	Clay/Rock Fill	100	28	118	
10	Rock/Clay Fill	100	28	118	
10	Rock Fill	0	38	118	
12	Blasted Zone	0	28	118	
13	Bedrock	0	Very Strang	125	

NOTE: Refer to Design Report for discussion of material zones and material strength parameters.

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS					
Soil No.	\$ (deg.)	C (0.s.f.)			
1	43.1	390			

	HARY OF I	
Soil No.	ø (deg.)	C (p.s.t.
2	31.6	72



FOR PERMIT REVIEW - 10/19/07

NOT FOR CONSTRUCTION

#### 1527+80UD.OUT

```
TABLE NO. 1
 COMPUTER PROGRAM DESIGNATION: UTEXAS4
Originally Coded By Stephen G. Wright
Version No. 4.0.2.0 - Last Revision Date: 1/29/2005
(C) Copyright 1985-2002 S. G. Wright - All rights reserved
 * RESULTS OF COMPUTATIONS PERFORMED USING THIS SOFTWARE
  SHOULD NOT BE USED FOR DESIGN PURPOSES UNLESS THEY HAVE
  BEEN VERIFIED BY INDEPENDENT ANALYSES, EXPERIMENTAL DATA
* OR FIELD EXPERIENCE. THE USER SHOULD UNDERSTAND THE ALGORITHMS
  AND ANALYTICAL PROCEDURES USED IN THIS SOFTWARE AND MUST HAVE
  READ ALL DOCUMENTATION FOR THIS SOFTWARE BEFORE ATTEMPTING
  TO USE IT. NEITHER SHINOAK SOFTWARE NOR STEPHEN G. WRIGHT
  MAKE OR ASSUME LIABILITY FOR ANY WARRANTIES, EXPRESSED OR
UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005
Licensed for use by: FMSM Engineers, FMSM Engineers
Time and date of run: Thu Oct 18 17:57:19 2007
Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final
Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UD.dat
  E W Brown Main Ash Pond Sta. 527+80
  Final Configuration 962.0'
TABLE NO. 3
*******
* NEW PROFILE LINE DATA *
---- Profile Line No. 1 - Material Type (Number): 1 --
Description: Gypsum
Point
               X
           -207.50
-27.50
27.50
                           902.00
                           962.00
    3
                           962.00
                           900.00
            213.50
                           900.00
            218.19
---- Profile Line No. 2 - Material Type (Number): 1 ---
Description: Starter Dike - Gypsum
Point X
          -258.50
                          885.00
   23
          -207.50
                          902.00
                          903.10
          -152.30
          -114.74
                          890.59
   5
             51.93
                          888.09
            151.94
                          889.59
   6
            194.65
                          890.59
---- Profile Line No. 3 - Material Type (Number): 2 ----
```

Page 1

```
Description: Fly Ash
 Point
              X
    1
          -340.00
                        885.00
           180.69
                        885.00
 ---- Profile Line No. 4 - Material Type (Number): 3 ---
Description: Existing Emb.
Point
          X
           110.89
                        870.00
   23
                        875.32
           124.19
           158.19
                        876.00
          218.19
238.19
                        900.00
                       900.00
870.20
870.20
   5
           312.69
328.78
                       870.90
           352.49
---- Profile Line No. 5 - Material Type (Number): 4 ----
Description: Old Working Platform
Point
            X
          105.89 868.00
110.89 870.00
350.69 870.00
   2
---- Profile Line No. 6 - Material Type (Number): 5 ----
Description: Bottom ASh
Point
         -340.00
                       868.00
          141.19
                       868.00
---- Profile Line No. 7 - Material Type (Number): 6 ----
Description: Foundation Soil
Point
           X
                       868.00
          141.19
          346.69
                       868.00
---- Profile Line No. 8 - Material Type (Number): 5 ----
Description: Bottom Ash
Point
             X
         141.19
                    868.00
  1
```

1527+80UD.OUT

```
148.69
                            864.00
                            864.00
             338.69
---- Profile Line No. 9 - Material Type (Number): 6 --
Description: Foundation Soil
Point
                X
                          832.38
           -340.00
                           838.19
           -165.72
                           844.00
    3
               8.56
                            849.02
             163.66
                            854.03
             318.75
                            870.90
             352.49 420.00
                            872.92
---- Profile Line No. 10 - Material Type (Number): 7 ----
Description: Bedrock
Point
            X
           -340.00
                            828.38
                            840.00
               8.69
             348.69
                            851.00
    3
                            851.00
             420.00
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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UD.dat
   E W Brown Main Ash Pond Sta. 527+80
   Final Configuration 962.0
* NEW MATERIAL PROPERTY DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
               ----- DATA FOR MATERIAL NUMBER 1 -----
Description: Gypsum
Unit weight of soil (material): 110.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - 0.0
Friction angle - - - - 35.00 (degrees)
No (zero) pore water pressures.
Negative pore water pressures are NOT allowed - set to zero.
                     --- DATA FOR MATERIAL NUMBER 2 -----
Description: Fly Ash
                                                  Page 3
```

## 1527+80UD.OUT

```
Unit weight of soil (material): 100.0
 CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
 Cohesion - - - - - - 0.0
Friction angle - - - - 32.00 (degrees)
 No (zero) pore water pressures.
 Negative pore water pressures are NOT allowed - set to zero.
            ----- DATA FOR MATERIAL NUMBER 3 -----
Description: Embankment
Unit weight of soil (material): 118.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - -
                        - 0.0
Friction angle - - - - 32.00 (degrees)
No (zero) pore water pressures.
Negative pore water pressures are NOT allowed - set to zero.
         ----- DATA FOR MATERIAL NUMBER 4 ------
Description: Working Platform
Unit weight of soil (material): 115.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - 0.0
Friction angle - - - - 35.00 (degrees)
Pore water pressures are defined by a piezometric line. Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.
   ----- DATA FOR MATERIAL NUMBER 5 ------
Description: Bottom Ash
Unit weight of soil (material): 115.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - 0.0
Friction angle - - - - 32.00 (degrees)
Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.
     ----- DATA FOR MATERIAL NUMBER 6 -----
Description: Foundation Soil
Unit weight of soil (material): 100.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - 0.0
```

```
1527+80UD.OUT
Friction angle - - - - 32.00 (degrees)
Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.
                                                 --- DATA FOR MATERIAL NUMBER 7 -----
 Description: Bedrock
Unit weight of soil (material): 125.0
 SHEAR STRENGTH IS VERY LARGE (INFINITE)
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Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UD.dat
       E W Brown Main Ash Pond Sta. 527+80
      Final Configuration 962.0
 1. (1) 1. (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1) . (1
 * NEW PIEZOMETRIC LINE.DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
                                  ---- Piezometric Line Number 1 ------
Unit weight of fluid (water): 62.4
 Point
                                     X
                                                                  870.00
                          -340.00
                                                                  870.00
                             110.89
                                                                  864.00
                             148.09
       3
                             338.69
                                                                  864.00
       4
                                                                   864.00
                             420.00
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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final
Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UD.dat
       E W Brown Main Ash Pond Sta. 527+80
       Final Configuration 962.0'
 TABLE NO. 16
 ******
 * NEW ANALYSIS/COMPUTATION DATA *
 Starting Center Coordinate for Search at -
                                                                                                                                             X: -200.00
                                                                                                                                             Y: 1100.00
```

Page 5

Required accuracy for critical center

1527+80UD.OUT

(= minimum spacing between grid points): 1.000

Critical shear surface not allowed to pass below Y: 847.00

For the initial mode of search circles are tangent to horizontal line at 
Y: 847.00

Radius: 253.00

only the LEFT face of the slope will be analyzed

Minimum weight required for computations to be performed: 975000

```
The following represent default values or values that were prevously defined:
 Subtended angle for slice subdivision: 3.00(degrees)
 There is no crack.
There is no water in a crack.
Conventional (single-stage) computations will be performed. Seismic coefficient: 0.000
Unit weight of water (or other fluid) in crack: 62.4
Automatic search output will be in long form.
Search will be continued after the initial mode to find a most critical circle.
Maximum number of trial grids for a given search mode: 50
No restrictions exist on the lateral extent of the search.
No shear surfaces other than the most critical will be saved for display later. Standard sign convention used for direction of shear stress on shear surface.
Procedure of Analysis: Spencer
Iteration limit: 100
Force imbalance: 1.000000e-005 (fraction of total weight)
Moment imbalance: 1.000000e-005 (fraction of moment due to total weight)
Initial trial factor of safety: 3.000
Initial trial side force inclination: 17.189 (degrees)
Minimum (most negative) side force inclination allowed in Spencer's procedure:
-10.00
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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80\D.dat
  E W Brown Main Ash Pond Sta. 527+80
  Final Configuration 962.0
```

These slope geometry were generated from the Profile Lines.

Point	×	Y
1 2 3 4 5 6 7 8	-340.00 -258.50 -207.50 -165.72 -152.30 -114.74 -27.50 8.56 8.69	885.00 885.00 902.00 915.93 920.40 932.92 962.00 962.00

```
1527+80UD.OUT
                               962.00
                  27.50
        10
                               953.86
                  51.93
        11
                               935.87
                 105.89
        12
                               934.20
                 110.89
        13
                               929.77
                 124.19
        14
                               924.10
        15
                 141.19
                               921.60
                 148.69
        16
                               920.52
                 151.94
        17
                               918.44
                 158.19
        18
                 163.66
                               916.61
        19
                               910.94
        20
                 180.69
                               906.28
        21 22
                 194.65
                 213.50
                               900.00
        23
                 218.19
                               900.00
                               900.00
        24
                 238.19
                              870.20
                 312.69
        25
                 318.75
                               870.20
        26
                              870.20
        27
                 328.78
        28
                 338.69
                               870.49
                              870.73
        29
                 346.69
                              870.79
                 348.69
        30
                              870.85
                 350.69
        31
                 352.49
                              870.90
        32
                              872.92
                 420.00
        33
Search will be conducted for LEFT face of slope
```

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Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UD.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0

TABLE NO. 30 \* OUTPUT FOR TYPE 1 AUTOMATIC SEARCH WITH CIRCLES \*

----- Output for Circles Tangent to a Given Horizontal Line --------- Tangent line elevation, Y: 847.00

Center Coor	dinates		1-Stage Factor of	Side Force Inclination			
×	Y	Radius	safety	(degrees)			Messages
-230.00	1070.00	223.00	Center r	ejected as	follows:	UTEXAS	NOTICE NUMBER
8060						circle	does not
intersect th	ne slope.						
-200.00 -170.00 -230.00 8060	1070.00 1070.00 1100.00	223.00 223.00 253.00	2.448 2.567 Center r	12.616 12.591 ejected as	follows:		NOTICE NUMBER
intersect th	ne slope.						
-200.00	1100.00	253.00	2.475	12.141 Page 7	4		

## 1527+80UD.OUT

CAUTION - THE FACTOR OF SAFETY COULD NOT BE COMPUTED FOR SOME OF THE GRID POINTS AROUND THE MINIMUM

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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final
Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80\D.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

#### TABLE NO. 33

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\* 1-STAGE FINAL CRITICAL CIRCLE INFORMATION \* \*\*\*\*\*\*\*\*\*\*\*\*\*

CAUTION - THE FACTOR OF SAFETY COULD NOT BE COMPUTED

FOR SOME OF THE GRID POINTS AROUND THE MINIMUM X Coordinate of Center . . . . -205.00Y Coordinate of Center 1091.00

226.00 Radius Factor of Safety . 2.351

15.35 Side Force Inclination (degrees) Number of Circles Tried . . 179

Number of Circles F Calculated for . . 113 Time Required for Search (seconds) .

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Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80\underballet

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

#### TABLE NO. 43

\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*\*

\* Coordinate, Weight, Strength and Pore Water Pressure \* Information for Individual Slices for Conventional \* Computations or First Stage of Multi-Stage Computations. \*

\* (Information is for the critical shear surface in the

Slice No.	X	ΥΥ	slice Weight	Matl. No.	Cohesion	Friction Angle	Pore Pressure
1	-297.95 -292.50	885.00 882.71	2499	2	0.0	32.00	0.0
2	-287.04 -281.48 -275.91	880.42 878.42 876.41	7331	2	0.0	32.00	0.0
3	-270.25 -264.58	874.70 873.00	11663	2	0.0	32.00	0.0
4	-261.54 -258.50	872.21 871.42	7779	- 2	0.0	32.00	0.0
5	-255.39 -252.28	870.71 870.00	9603	2	0.0	32.00	0.0
6	-246.99	869.00	21382	Page 20	0.0	32.00	0.0

			152	7+80UD.	DUT		
7	-241.70 $-235.84$	868.00 867.19	30753	5	0.0	32.00	175.2
8	-229.98 -224.09	866.38 865.89	37787	5	0.0	32.00	256.8
9	-218.19 -212.85	865.39 865.20	39513	5	0.0	32.00	299.5
	-207.50 -206.25	865.01 865.01	9900	5	0.0	32.00	311.6
10	-205.00	865.00 865.15	49745	5	0.0	32.00	302.3
11	-199.09 $-193.17$	865.31	53877	5	0.0	32.00	263.7
12	-187.27 $-181.38$	865.77 866.24		5	0.0	32.00	186.6
13	-175.51 $-169.65$	867.01 867.78	56973		0.0	32.00	131.6
14	-168.97 -168.30	867.89 868.00	6730	5			0.0
15	-167.01 $-165.72$	868.22 868.44	12977	2	0.0	32.00	
16	-161.72 -157.72	869.22 870.00	40991	2	0.0	32.00	0.0
17	-155.01 -152.30	870.62 871.23	28385	2	0.0	32.00	0.0
18	-146.59 -140.87	872.76 874.29	60892	2	0.0	32.00	0.0
19	-135.24	876.12 877.94	60858	2	0.0	32.00	0.0
20	-129.62 -124.09	880.06	59899	2	0.0	32.00	0.0
21	-118.57 -116.65	882.18 882.99	20684	2	0.0	32.00	0.0
22	-114.74 $-113.39$	883.81 884.40	14481	2	0.0	32.00	0.0
23	-112.05 $-106.72$	885.00 887.57	56275	1	0.0	35.00	0.0
24	-101.39 $-101.17$	890.15 890.27	2367	1	0.0	35.00	0.0
25	-100.94 -95.76	890.38 893.24	52409	1	0.0	35.00	0.0
26	-90.58 -85.56	896.10 899.23	47963	1	0.0	35.00	0.0
27	-80.54 -75.69	902.36 905.75	42894	1	0.0	35.00	0.0
28	-70.84 -66.17	909.13 912.77	37317	1	0.0	35.00	0.0
	-61.50 -57.03	916.40 920.28	31354	1	0.0	35.00	0.0
29	-52.56	924.15 928.26	25137	1	0.0	35.00	0.0
30	-48.30 -44.04	932.36	18807	1	0.0	35.00	0.0
31	-40.00 -35.96	936.68 941.00	12508	1	0.0	35.00	0.0
32	-32.15 -28.34	945.53 950.05			0.0	35.00	0.0
33	-27.92 -27.50	950.58 951.11	1038	1	0.0	35.00	0.0
34	-23.96 -20.42	955.85 960.59	4789	1			0.0
35	-19.93 -19.43	961.29 962.00	77	1	0.0	35.00	0.0

No water in crack.

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Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80Up.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0

TABLE NO. 44

\*\*\*\*\*\*\*\*\*\*\* Seismic Forces and Forces Due to Distributed Loads for \* Individual Slices for Conventional Computations or the \* First Stage of Multi-Stage Computations.
(Information is for the critical shear surface in the \* case of an automatic search.) \*

There are no seismic forces or forces due to distributed loads for the current shear surface UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005 Licensed for use by: FMSM Engineers, FMSM Engineers
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Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80\UD.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

TABLE NO. 47

\*\*\*\*\*\*\*\*\*\*\*\*

Information for the Iterative Solution for the Factor of Safety and Side Force Inclination by Spencer's Procedure

Allowable force imbalance for convergence: Allowable moment imbalance for convergence: 1402

Trial Trial Factor Side Force Force Iter- of Inclination Imbalance ation Safety (degrees) (lbs.)	Moment Imbalance Delta-F (ftlbs.)	Delta Theta (degrees)
First-order corrections to F and Theta	-5.069e+007 	-1.3719 -0.8428
	1.302e+007 0.1565 -0.1495	-0.9207 -0.9964
	4.737e+004 	0.0011 0.0011
4 2.35102 15.3506 2.103e-006 - First-order corrections to F and Theta	-0.0000	0.000
UTEXAS4 S/N:00147 - Version: 4.0.2.0 - L Licensed for use by: FMSM Engineers, FMSM Time and date of run: Thu Oct 18 17:57:19 Name of input data file: J:\Jobs\2006proj Pag	Engineers 2007	

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E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

TABLE NO. 55 \* Check of Computations by Spencer's Procedure (Results are for the \* \* critical shear surface in the case of an automatic search.)

Summation of Horizontal Forces: 7.63176e-011

Summation of Vertical Forces: 8.09592e-011

Summation of Moments: -2.97950e-008

Mohr Coulomb Shear Force/Shear Strength Check Summation: 3.34914e-011 UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005
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Time and date of run: Thu Oct 18 17:57:19 2007
Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final
Design\E2.18 Slope Stability\slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UD.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

TABLE NO. 58 

SPENCER'S PROCEDURE USED TO COMPUTE THE FACTOR OF SAFETY Factor of Safety: 2.351 Side Force Inclination: 15.35

-11-2	VAI	LUES AT CENTER	Total Normal	OF SLICE Effective Normal	shear
Slice No.	x-Center	Y-Center	Stress	Stress	Stress
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19	-292.50 -281.48 -270.25 -261.54 -255.39 -246.99 -235.84 -224.09 -212.85 -206.25 -199.09 -187.27 -175.51 -168.97 -167.01 -161.72 -155.01 -146.59 -135.24	882.71 878.42 874.70 872.21 870.71 869.00 867.19 865.20 865.20 865.01 865.77 867.01 867.89 868.22 869.22 870.62 872.76 876.12	327.3 898.8 1347.6 1624.4 1920.4 2451.5 3054.8 3609.2 4042.3 4260.4 4447.3 4696.9 4858.6 4920.9 4938.0 4965.3 4985.8 4971.3	327.3 898.8 1347.6 1624.4 1920.4 2451.5 2879.7 3352.5 3742.7 3948.9 4145.0 4433.2 4672.0 4789.4 4938.0 4965.3 4985.8 4971.3 4905.3	87.0 238.9 358.2 431.7 510.4 651.6 765.4 891.0 994.8 1049.6 1101.7 1178.3 1241.8 1272.9 1312.5 1319.7 1325.2 1321.3 1303.8
			Pa	ge 23	

				1527+80	UD.OUT	
20		-124.09	880.06	4780.2	4780.2	1270.5
21		-116.65	882.99	4670.6	4670.6	1241.4
22		-113.39	884.40	4611.8	4611.8	1225.8
23		-106.72	887.57	4421.1	4421.1	1316.7
24	4	-101.17	890.27	4273.9	4273.9	1272.9
25		-95.76	893.24	4099.4	4099.4	1220.9
26		-85.56	899.23	3746.4	3746.4	1115.8
27		-75.69	905.75	3355.0	3355.0	999.2
28		-66.17	912.77	2930.8	2930.8	872.9
29		-57.03	920.28	2479.4	2479.4	738.5
30		-48.30	928.26	2007.2	2007.2	597.8
31		-40.00	936.68	1520.7	1520.7	452.9
32		-32.15	945.53	1027.2	1027.2	305.9
33		-27.92	950.58	756.6	756.6	225.3
34		-23.96	955.85	401.6	401.6	119.6
35		-19.93	961.29	44.7	44.7	13.3

UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005

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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UD.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

TABLE NO. 59

## ---- VALUES AT RIGHT SIDE OF SLICE ----

slice No.	X-Right	Side Force	Y-Coord. of Side Force Location	Fraction of Height	Sigma at Top	Sigma at Bottom
1 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24	-287.04 -275.91 -264.58 -258.50 -252.28 -241.70 -229.98 -218.19 -207.50 -205.00 -193.17 -181.38 -169.65 -168.30 -165.72 -157.72 -157.72 -152.30 -140.87 -129.62 -118.57 -114.74 -112.05 -101.39 -100.94	2539 9030 18013 23383 29512 41742 56161 70796 83381 86163 98247 108138 115463 116131 117389 120298 121390 121280 117912 111462 108515 106228 97178 96734	884.21 882.00 880.70 880.23 879.78 879.10 878.85 879.07 879.69 879.89 881.13 882.81 884.93 885.20 885.72 887.49 888.82 891.97 895.56 899.58 901.09 902.20 906.62	0.827 0.651 0.642 0.648 0.573 0.491 0.414 0.397 0.394 0.381 0.373 0.366 0.365 0.365 0.364 0.352 0.352 0.352 0.352 0.352	1582.0 1931.6 2675.6 3138.8 2394.9 1686.6 1269.0 1000.2 827.7 793.9 659.1 553.7 466.8 457.6 437.1 380.9 348.5 294.0 257.6 239.7 238.3 239.1 185.3 183.2	-513.3 96.5 218.3 183.0 938.6 1875.7 2582.7 3131.0 3520.3 3598.5 3910.5 4136.3 4287.8 4301.0 4330.5 4393.8 4412.9 4391.6 4289.0 4106.4 4022.9 3957.7 3783.7

			1527+80UD	.OUT		
25	-90.58	85527	911.63	0.346	140.5	3535.8
	-80.54	72845	916.79	0.344	105.6	3242.6
26		59337	922.27	0.342	76.4	2902.0
27	-70.84			0.340	51.3	2520.5
28	-61.50	45688	928.05			2103.7
29	-52.56	32610	934.12	0.338	28.7	
30	-44.04	20812	940.44	0.335	8.2	1655.3
31	-35.96	10980	947.02	0.331	-7.5	1172.3
and the second		3755	954.00	0.339	9.7	610.9
32	-28.34		954.95	0.353	32.0	520.8
33	-27.50	3122	20 20 12 C		CV 144	55.8
34	-20.42	52	961.16	0.404	15.0	
35	-19.43	0	962.00	0.000	0.0	0.0

Read end-of-file on input while looking for another command word. End of input data assumed - normal termination.

## 1527+80UDseismic.OUT

```
TABLE NO. 1
COMPUTER PROGRAM DESIGNATION: UTEXAS4
Originally Coded By Stephen G. Wright
Version No. 4.0.2.0 - Last Revision Date: 1/29/2005
(C) Copyright 1985-2002 S. G. Wright - All rights reserved
* RESULTS OF COMPUTATIONS PERFORMED USING THIS SOFTWARE
   SHOULD NOT BE USED FOR DESIGN PURPOSES UNLESS THEY HAVE
   BEEN VERIFIED BY INDEPENDENT ANALYSES, EXPERIMENTAL DATA
* OR FIELD EXPERIENCE. THE USER SHOULD UNDERSTAND THE ALGORITHMS
* AND ANALYTICAL PROCEDURES USED IN THIS SOFTWARE AND MUST HAVE
* READ ALL DOCUMENTATION FOR THIS SOFTWARE BEFORE ATTEMPTING
* TO USE IT. NEITHER SHINOAK SOFTWARE NOR STEPHEN G. WRIGHT
* MAKE OR ASSUME LIABILITY FOR ANY WARRANTIES, EXPRESSED OR
UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005
Licensed for use by: FMSM Engineers, FMSM Engineers
Time and date of run: Thu Oct 18 17:57:19 2007
Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UDseismic.dat
   E W Brown Main Ash Pond Sta. 527+80
   Final Configuration 962.0'
TABLE NO. 3
* NEW PROFILE LINE DATA *
*******
---- Profile Line No. 1 - Material Type (Number): 1 ----
Description: Gypsum
Point
                 X
            -207.50
-27.50
27.50
                               902.00
                               962.00
                               962.00
                               900.00
              213.50
              218.19
                               900.00
---- Profile Line No. 2 - Material Type (Number): 1 ----
Description: Starter Dike - Gypsum
                  X
Point
                               885.00
            -258.50
    23
            -207.50
                               902.00
                               903.10
            -152.30
                               890.59
             -114.74
              51.93
151.94
                               888.09
    5
                               889.59
                               890.59
              194.65
```

```
1527+80UDseismic.OUT
---- Profile Line No. 3 - Material Type (Number): 2 ----
Description: Fly Ash
Point
         -340.00
                      885.00
                      885.00
          180.69
---- Profile Line No. 4 - Material Type (Number): 3 ----
Description: Existing Emb.
Point X
                    870.00
          110.89
   2
          124.19
                     875.32
          158.19
                    876.00
          218.19
                      900.00
   5
                      900.00
          238.19
                     870.20
870.20
   6
          312.69
          328.78
352.49
                      870.90
---- Profile Line No. 5 - Material Type (Number): 4 ---
Description: Old Working Platform
Point
            X
          105.89
                     868.00
                     870.00
          110.89
          350.69
                     870.00
---- Profile Line No. 6 - Material Type (Number): 5 ----
Description: Bottom ASh
Point
            X
        -340.00
                     868.00
                     868.00
         141.19
---- Profile Line No. 7 - Material Type (Number): 6 ----
Description: Foundation Soil
Point
           X
         141.19
                     868.00
                   868.00
        346.69
---- Profile Line No. 8 - Material Type (Number): 5 ----
Description: Bottom Ash
Point X
```

```
1527+80UDseismic.OUT
                            868.00
             141.19
             148.69
                            864.00
                            864.00
             338.69
  ---- Profile Line No. 9 - Material Type (Number): 6 ----
Description: Foundation Soil
Point
                            832.38
            -340.00
           -165.72
8.56
                            838.19
    23
                            844.00
                            849.02
    4
             163.66
                            854.03
             318.75
    6
             352.49
                            870.90
                            872.92
             420.00
---- Profile Line No. 10 - Material Type (Number): 7 ----
Description: Bedrock
Point
                X
           -340.00
                          828.38
                      840.00
    2
               8.69
            348.69
                            851.00
    3
                            851.00
             420.00
UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005
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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final
Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @
962.0\1527+80UDseismic.dat
  E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'
TABLE NO. 4
* NEW MATERIAL PROPERTY DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
**************************
              ----- DATA FOR MATERIAL NUMBER 1 -----
Description: Gypsum
unit weight of soil (material): 110.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - 0.0
Friction angle - - - - 35.00 (degrees)
No (zero) pore water pressures.
Negative pore water pressures are NOT allowed - set to zero.
            ----- DATA FOR MATERIAL NUMBER 2 -----
```

Page 3

```
Description: Fly Ash
Unit weight of soil (material): 100.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - 0.0
Friction angle - - - - 32.00 (degrees)
No (zero) pore water pressures.
Negative pore water pressures are NOT allowed - set to zero.
                  -- DATA FOR MATERIAL NUMBER 3 -----
Description: Embankment
Unit weight of soil (material): 118.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - 0.0
Friction angle - - - - 32.00 (degrees)
No (zero) pore water pressures.
Negative pore water pressures are NOT allowed - set to zero.
   ----- DATA FOR MATERIAL NUMBER 4 -----
Description: Working Platform
Unit weight of soil (material): 115.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - 0.0
Friction angle - - - - 35.00 (degrees)
Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.
   ----- DATA FOR MATERIAL NUMBER 5 ------
Description: Bottom Ash
Unit weight of soil (material): 115.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - 0.0
Friction angle - - - - 32.00 (degrees)
Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.
   ----- DATA FOR MATERIAL NUMBER 6 -----
Description: Foundation Soil
Unit weight of soil (material): 100.0
```

```
1527+80uDseismic.OUT
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - 0.0
Friction angle - - - - 32.00 (degrees)
Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.
       ----- DATA FOR MATERIAL NUMBER 7 ------
Description: Bedrock
Unit weight of soil (material): 125.0
SHEAR STRENGTH IS VERY LARGE (INFINITE)
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   E W Brown Main Ash Pond Sta. 527+80
  Final Configuration 962.0
TABLE NO. 6
*************************
* NEW PIEZOMETRIC LINE DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
************************
     ----- Piezometric Line Number 1 -----
Unit weight of fluid (water): 62.4
Point
                X
                           870.00
          -340.00
           110.89
                           870.00
  2
                           864.00
   3
           148.09
  4
           338.69
                           864.00
                           864.00
           420.00
UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005
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Name of input data file: j:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UDseismic.dat
  E W Brown Main Ash Pond Sta. 527+80
  Final Configuration 962.0'
TABLE NO. 16
***********
* NEW ANALYSIS/COMPUTATION DATA *
***********
```

Page 5

Starting Center Coordinate for Search at -

1527+80UDseismic.OUT

x: -200.00 Y: 1100.00

Required accuracy for critical center (= minimum spacing between grid points): 1.000

Critical shear surface not allowed to pass below Y: 847.00 For the initial mode of search circles are tangent to horizontal line at -

Y: 847.00 Radius: 253.00

only the LEFT face of the slope will be analyzed

Minimum weight required for computations to be performed: 975000 Seismic coefficient: 0.100

Seismic force acts at center of gravity.

The following represent default values or values that were prevously defined: Subtended angle for slice subdivision: 3.00(degrees) There is no crack. There is no water in a crack. Conventional (single-stage) computations will be performed.
Unit weight of water (or other fluid) in crack: 62.4
Automatic search output will be in long form.
Search will be continued after the initial mode to find a most critical circle.
Maximum number of trial grids for a given search mode: 50
No restrictions exist on the lateral extent of the search. No shear surfaces other than the most critical will be saved for display later. Standard sign convention used for direction of shear stress on shear surface. Procedure of Analysis: Spencer

Iteration limit: 100
Force imbalance: 1.000000e-005 (fraction of total weight)
Moment imbalance: 1.000000e-005 (fraction of moment due to total weight)
Initial trial factor of safety: 3.000

Initial trial side force inclination: 17.189 (degrees)

Minimum (most negative) side force inclination allowed in Spencer's procedure: -10.00

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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final
Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @
962.0\1527+80UDseismic.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

TABLE NO. 26 \*\*\*\*\*\*\* \* NEW, COMPUTED SLOPE GEOMETRY DATA \*

These slope geometry were generated from the Profile Lines.

Point	×	Y
1	-340.00	885.00
3	-258.50 -207.50	885.00 902.00

```
1527+80UDseismic.OUT
                                   915.93
                  -165.72
           5
                  -152.30
                                   920.40
                  -114.74
           6
                                   932.92
          7
                   -27.50
                                   962.00
                                   962.00
           8
                      8.56
                                   962.00
          9
                      8.69
                                   962.00
         10
                     27.50
                     51.93
                                  953.86
         11
                                  935.87
         12
                   105.89
                                   934.20
                   110.89
         13
                                   929.77
                   124.19
         14
                   141.19
                                   924.10
         15
         16
                   148.69
                                   921.60
                                   920.52
         17
                   151.94
                                   918.44
                   158.19
         18
                                  916.61
         19
                   163.66
                                  910.94
         20
                   180.69
         21
                                   906.28
                   194.65
         22
                                   900.00
                   213.50
         23
                                   900.00
                   218.19
         24
                   238.19
                                   900.00
         25
                   312.69
                                  870.20
         26
                                  870.20
                   318.75
                                  870.20
         27
                   328.78
         28
                   338.69
                                  870.49
         29
                                  870.73
                   346.69
                                  870.79
         30
                   348.69
                                  870.85
         31
                   350.69
         32
                   352.49
                                  870.90
                                  872.92
         33
                   420.00
search will be conducted for LEFT face of slope
UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005
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Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final
Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown slope
Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @
962.0\1527+80UDseismic.dat
  E W Brown Main Ash Pond Sta. 527+80
  Final Configuration 962.0
TABLE NO. 30
*******************************
* OUTPUT FOR TYPE 1 AUTOMATIC SEARCH WITH CIRCLES *
********************************
----- Output for Circles Tangent to a Given Horizontal Line -----
----- Tangent line elevation, Y: 847.00
                                        1-Stage
Center Coordinates
                                        Factor
                                                  Side Force
                                           of
                                                  Inclination
                                                                                            Messages
                                                    (degrees)
                                                                   Iterations
                             Radius
                                        safety
      X
                                                                                 UTEXAS NOTICE NUMBER
                              223.00 center rejected as follows:
                1070.00
    -230.00
                                                                                 circle does not
intersect the slope.
                                                                         5
                                                      16,900
                                         1.803
```

Page 7

8060

-200.00

1070.00

223.00

-215.00 9100	1112.00	1527+80 247.00 Center r	UDseismic.OUT ejected as follows	s:	UTEXAS ERROR NUMBER
(all slices	) is less th	an the acceptabl	e minimum		Total weight of soil
number 1.	Total weight	of all slices:	9.70456e+005		weight for stage
weight:	9.75000e+005				Minimum acceptable
-213.00 -215.00 9100	1112.00 1113.00	247.00 1.753 248.00 Center r	19.598 ejected as follows	5	UTEXAS ERROR NUMBER
(all slices	) is less th	an the acceptable	e minimum		Total weight of soil
number 1.	rotal weight	of all slices:	9.74057e+005		weight for stage
weight:	9.75000e+005				Minimum acceptable
-213.00 -215.00 -214.00 -213.00 Critic x: -214.00	New 9-Point 1114.00 1114.00 1114.00 cal Circle At Y: 1113.0	248.00 1.753 Grid (only new p 249.00 1.753 249.00 1.753 249.00 1.753 Fter the Current 00 Radius: 24	19.534 19.593 points calculated) 19.468 19.529 19.589 Mode of Search 18.000 nclination: 19.53	5 5 5 	4.5.5
CAUTION - THE FOR SOME OF ***********************************	HE FACTOR OF THE GRID POI ************  00147 - Ver * use by: FMS te of run: The thata file: Slope Stab	SAFETY COULD NOT INTS AROUND THE M ************* sion: 4.0.2.0 - IM Engineers, FMS IU Oct 18 17:57:1 J:\Jobs\2006pro ility\Slope Stab 59+09 new Sta. 5	INIMUM ***********************************	*** 1/29/20 W. Brow	
E W Brown Final Conf	Main Ash Pon iguration 96	d Sta. 527+80 2.0'			
	***	**************************************			
FOR SOME OF X Coordinate Y Coordinate Radius Factor of Sa Side Force I Number of Ci Number of Ci Time Required UTEXAS4 S/N:0	THE GRID POI of Center. of Center. fety nclination (or cles Tried rcles F Calculus for Search	degrees)	INIMUM214.001113.00248.001.75319.5314282820.4 Latest Revision: 1	/29/200	05
		4 Engineers, FMSM		1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	

Time and date of run: Thu Oct 18 17:57:19 2007

Name of input data file: J:\Jobs\2006proj\Lx2006193 - E. W. Brown Main Pond Final Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UDseismic.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

TABLE NO. 43 \* Coordinate, Weight, Strength and Pore Water Pressure \* Information for Individual Slices for Conventional \* Computations or First Stage of Multi-Stage Computations. \* \* (Information is for the critical shear surface in the 

slice No.	X	Y	Slice Weight	Matl. No.	Cohesion	Friction Angle	Pore Pressure
1	-311.57 -305.54	885.00 882.60	2892	2	0.0	32.00	0.0
2	-299.50 $-293.35$	880.21 878.13	8453	2	0.0	32.00	0.0
3	-287.20 $-280.95$	876.05 874.30	13380	2	0.0	32.00	0.0
4	-274.70 -269.13	872.54 871.27	15313	2	0.0	32.00	0.0
5	-263.55 -261.02	870.00 869.51	7818	2	0.0	32.00	0.0
6	-258.50 -255.48	869.03 868.51	10631	2	0.0	32.00	0.0
7	-252.46 -246.02	868.00 867.16	29020	5	0.0	32.00	177.1
8	-239.58 -233.11	866.32 865.82	37300	5	0.0	32.00	260.7
	-226.64 -220.32	865.32 865.16	43302	5	0.0	32.00	301.9
9	-214.00 -210.75	865.00 865.04	24641	5	0.0	32.00	309.3
10	-207.50	865.09	53210	5	0.0	32.00	285.5
11	-201.02 -194.53	865.43 865.77	57604	5	0.0	32.00	221.9
12	-188.08 -181.62	866.44 867.12		5	0.0	32.00	152.2
13	-178.58 -175.54	867.56 868.00	28455	2	0.0	32.00	0.0
14	-170.63 -165.72	868.87 869.74	47487	2	0.0	32.00	0.0
15	-165.09 -164.45	869.87 870.00	6261	2	0.0	32.00	0.0
16	-158.38 $-152.30$	871.40 872.80	61141	2	0.0	32.00	0.0
17	-146.06 $-139.81$	874.58 876.36	64496				0.0
18	-133.67 -127.53	878.46 880.56	64255	2	0.0	32.00	0.0
19	-121.98 -116.43	882.78 885.00	58029	2	0.0	32.00	0.0
20	-115.58 -114.74	885.37 885.73	8786	1	0.0	35.00	0.0
21	-109.67 -104.59	888.08 890.44	51936	1	0.0	35.00	0.0

Page 18

			1527+80	OUDseism	ic.OUT		
22	-98.84	893.45	56619	1	0.0	35.00	0.0
23	-93.09 -87.51 -81.93	896.47 899.78 903.09	51855	1	0.0	35.00	0.0
24	-76.53 -71.12	906.69 910.29	46311	1	0.0	35.00	0.0
25	-65.92 -60.71	914.17 918.05	40118	1	0.0	35.00	0.0
26	-55.71	922.19	33419	1	0.0	35.00	0.0
27	-50.72 -45.94	926.34 930.74	26368	1	0.0	35.00	0.0
28	-41.17 -36.64	935.14 939.78	19130	1	0.0	35.00	0.0
29	-32.10 -29.80	944.43 946.98	7212	1	0.0	35.00	0.0
30	-27.50 -23.35	949.53 954.52	6825	1	0.0	35.00	0.0
31	-19.20 -18.23	959.52 960.76	264	1	0.0	35.00	0.0
	-17.27	962.00					

No water in crack.

UTEXAS4 S/N:00147 - Version: 4.0.2.0 - Latest Revision: 1/29/2005

Licensed for use by: FMSM Engineers, FMSM Engineers
Time and date of run: Thu Oct 18 17:57:19 2007
Name of input data file: J:\Jobs\2006proj\LX2006193 - E. W. Brown Main Pond Final Design\E2.18 Slope Stability\Slope Stability - Phase Embankments\E.W. Brown Slope Stability (Phase Emb.)\559+09 new Sta. 527+80\KA Check Crest @ 962.0\1527+80UDseismic.dat

E W Brown Main Ash Pond Sta. 527+80 Final Configuration 962.0'

#### TABLE NO. 44

- the North of the N Seismic Forces and Forces Due to Distributed Loads for \* Individual Slices for Conventional Computations or the \* First Stage of Multi-Stage Computations. \* (Information is for the critical shear surface in the \*

- \* case of an automatic search.) \*

#### FORCES DUE TO DISTRIBUTED LOADS

			Y for				
Slices No.	×	Seismic Force	Seismic Force	Normal Force	Shear Force	×	Y
1 2 3 4 5 6 7 8 9 10 11 12 13	-305.54 -293.35 -280.95 -269.13 -261.02 -255.48 -246.02 -233.11 -220.32 -210.75 -201.02 -188.08 -178.58	-289 -845 -1338 -1531 -782 -1063 -2902 -3730 -4330 -2464 -5321 -5760 -2846	883.80 881.56 879.65 878.14 877.26 877.31 878.27 879.78 881.63 883.21 885.08 887.84	000000000000000000000000000000000000000	000000000000000000000000000000000000000	-305.54 -293.35 -280.95 -269.13 -261.02 -255.48 -246.02 -233.11 -220.32 -210.75 -201.02 -188.08 -178.58	882.60 878.13 874.30 871.27 869,51 868.51 867.16 865.82 865.16 865.43 866.44 867.56
14 15	-170.63 -165.09	-4749 -626	892.07 893.48	0	0	-170.63 -165.09	868.87 869.87
				Page 10			

# 3. Evaluation Methodology

Seed and Idriss (1971) proposed a simplified procedure for evaluating liquefaction potential based on soil type, relative density or void ratio (indicated by penetration resistance), initial confining stress, and the intensity of ground shaking. This methodology has undergone continued development and refinement since its initial inception, and was evaluated in detail and summarized by a workshop of experts organized by the National Center for Earthquake Engineering Research (NCEER) (Youd and Idriss 1997). This updated liquefaction assessment method, as summarized by Youd et al. (2001), is often referred to as the "NCEER Method". The NCEER methodology for evaluating liquefaction resistance based on cone penetration test (CPT) results, as outlined in this section, was adopted for evaluating the ash basin deposits.

The NCEER method compares the liquefaction resistance of a soil, expressed in terms of a cyclic resistance ratio (CRR), to the cyclic stress ratio (CSR) induced by the design earthquake. Both the CRR and CSR represent shear stress normalized with respect to the vertical effective stress in the soil. The factor of safety against liquefaction (FS<sub>liq</sub>) is defined as the ratio of CRR to CSR:

$$FS_{liq} = \frac{CRR}{CSR}$$

For this project, the CRR was determined from the CPT tip resistance (Section 3.2) and the CSR was estimated using the simplified method (Section 3.1).

# 3.1. Cyclic Stress Ratio (CSR)

The cyclic stress ratio (CSR) induced by an earthquake is the average shear stress ratio, assumed to be 65% of the maximum induced shear stress ratio. The CSR at depth in a soil profile can be computed in a site response analysis, often accomplished using one-dimensional, equivalent-linear ground response methods.

In this study, the CSR was estimated using the simplified equation for CSR (Seed and Idriss 1971):

$$CSR = 0.65 \left(\frac{a_{max}}{g}\right) \left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) r_d$$

Where  $a_{max}$  is the peak horizontal acceleration that would occur at the ground surface (in the absence of liquefaction) due to an earthquake, g is the acceleration of gravity,  $\sigma_{vo}$  and  $\sigma'_{vo}$  are the total and effective vertical overburden stress, and  $r_d$  is a stress reduction coefficient that accounts for flexibility in the soil profile.

Equation 2 yields approximate values of CSR that are limited in accuracy by the definition of the stress reduction coefficient ( $r_d$ ). The range in computed values of  $r_d$  are plotted versus depth in Figure 3. The NCEER workshop (Youd et al. 2001) suggests estimating average values of  $r_d$  using:

For 
$$z \le 9.15 \,\text{m}$$
  $r_d = 1.0 - 0.00765 \,z$   
For  $9.15 < z \le 23 \,\text{m}$   $r_d = 1.174 - 0.0267 \,z$  3

Where z is the depth below the ground surface in meters. More recent research (Seed et al. 2003) indicates that values of r<sub>d</sub> can vary more widely with depth than shown in Figure 3.

## 3.2. Cyclic Resistance Ratio (CRR)

The cyclic resistance ratio (CRR) of the soil, or the shear stress required to cause liquefaction under field conditions, was determined from the measured CPT tip resistance and the NCEER field performance correlation (Robertson and Wride 1998; Youd et al. 2001).

Field measurements of CPT tip stress were first normalized and adjusted to account for the effects of overburden pressure and fines content, as described in Section 3.2.1 and Section 3.2.2, respectively. The cyclic resistance ratio for a magnitude 7.5  $M_w$  earthquake (CRR<sub>7.5</sub>) was estimated using the correlation suggested by Robertson and Wride (1998; Youd et al. 2001) and shown in Figure 4. The following equations represent the CPT-based clean sand base curve, and can be used to compute CRR<sub>7.5</sub>:

For 
$$(q_{c1N})_{cs} < 50$$
  $CRR_{7.5} = 0.833 \left( \frac{(q_{c1N})_{cs}}{1,000} \right) + 0.05$   
For  $50 \le (q_{c1N})_{cs} < 160$   $CRR_{7.5} = 93 \left( \frac{(q_{c1N})_{cs}}{1,000} \right)^3 + 0.08$ 

Where  $(q_{c1N})_{cs}$  is the clean-sand equivalent penetration resistance normalized to one atmosphere of overburden pressure.

To account for other earthquake magnitudes,  $CRR_{7.5}$  is modified using a magnitude scaling factor (MSF, Section 3.2.3). Adjustments are also made to account for high overburden pressures ( $K_{\alpha}$ , Section 3.2.4) and static driving shear stresses ( $K_{\alpha}$ , Section 3.2.5). The CRR is then computed as:

$$CRR = CRR_{7.5} \cdot MSF \cdot K_{\sigma} \cdot K_{\alpha}$$

# 3.2.1. Normalized, Corrected Cone Penetration Resistance (qc1N)

The measured resistance to penetration of the conical CPT tip  $(q_c)$ , in units of stress, is recorded as the cone penetrometer probe is advanced into the soil. Due to details of the probe design, pore water pressures acting behind the cone tip affect the measurement of tip resistance. A corrected cone tip resistance  $(q_t)$  is computed as:

$$q_t = q_c + (1 - A) \cdot u_2 \tag{6}$$

Where A is the net area ratio (0.85 for the probe used here) and  $u_2$  is the pore water pressure measured just behind the cone tip. The effect of this correction is more significant in

There have been many suggestions for the relationship between MSF and earthquake magnitude (Figure 7). Considering the range in proposed magnitude scaling factors, the NCEER workshop (Youd et al. 2001) recommended a lower bound curve to conservatively predict lower values of CRR for  $M_W$ <7.5. The lower bound curve was defined by Idriss with this equation:

$$MSF = \frac{10^{2.24}}{M_{W}^{2.56}}$$

The NCEER workshop (Youd et al. 2001) also recommend this equation for computing MSF for  $M_W > 7.5$ .

## 3.2.4. Correction for Overburden Pressure (K<sub>σ</sub>)

The simplified procedure was developed using data from liquefied soil deposits less than about 45 ft deep. Seed (1983) proposed using an overburden correction factor ( $K_{\sigma}$ ) to adjust the CRR for higher consolidation stresses.

Laboratory data show a nonlinear increase in liquefaction resistance with an increase in effective confining pressure. Hynes and Olsen (1999) developed a set of curves relating  $K_{\sigma}$  to vertical effective stress for soils of different relative densities. The NCEER workshop (Youd et al. 2001) endorsed the relationships in Figure 8 for use in engineering practice for clean sands, silty sands, and gravels. The overburden correction factors in Figure 8 can be expressed:

$$K_{\sigma} = \left(\frac{\sigma'_{VO}}{P_{\theta}}\right)^{f-1} \le 1$$

Where  $P_a$  is atmospheric pressure in the units of the vertical effective stress,  $\sigma'_{vo}$ . Using linear interpolation, the f coefficient is related to relative density  $D_r$  (decimal) with:

$$0.6 \le (f = 1.0 - 0.5 \cdot D_r) \le 0.8$$

The upper bound curve (f=0.8) is assigned for relative densities less than or equal to 40%, and the lower bound curve (f=0.6) is assigned for relative densities greater than or equal to 80%.

## 3.2.5. Correction for Static Shear Stress (Kα)

Most of the data used to develop the simplified procedure was from sites with a level to gently sloping surface. Seed (1983) proposed a correction to adjust the CRR for static shear stress imposed by a sloping ground surface. The static shear stress correction factor  $(K_{\alpha})$  permits the consideration of more steeply sloping ground conditions.

A sloping ground surface creates driving shear stresses in the soil mass under static conditions. The static driving shear stress can be represented by  $\alpha$ , which is the ratio of the static driving shear stress acting on a horizontal plane ( $\tau_{hv}$ ) to the vertical effective stress ( $\sigma'_{vo}$ ):

$$\alpha = \frac{\tau_{hv}}{\sigma'_{vo}}$$

Static shear increases the cyclic resistance of a dense soil under low confining pressure, because higher induced stresses are required to cause stress reversals. However, loose soils and some soils under high confining pressures have lower cyclic resistance due to strain softening behavior. The  $K_{\alpha}$  parameter, which is used to modify the CRR for this effect, can vary widely depending upon the value of alpha and the relative density of the soil (see Figure 9 from Harder and Boulanger 1997). The NCEER workshop (Youd et al. 2001) reviewed the available data, but did not make a consensus recommendation on the selection of  $K_{\alpha}$  values. Because  $K_{\alpha}$  is poorly defined,  $K_{\alpha}$  is often assumed to be one (no adjustment in CRR) in liquefaction assessments.

Conservative  $K_{\alpha}$  values can be assigned for soils that are expected to behave contractively (i.e., soils with low relative density). In this case,  $K_{\alpha}$  is set to one for relative densities greater than 40%. For relative densities less than or equal to 40%, a straight line relationship through the range indicated in Figure 9 can be assumed. In equation form:

$$K_{\alpha} = \begin{cases} 1 - 2\alpha & \text{for } D_R \le 40\% \\ 1 & \text{for } D_R > 40\% \end{cases}$$
 18

# 4. Liquefaction Assessment

## 4.1. Critical Section for Analysis

Preliminary slope stability analyses were performed assuming that liquefaction would occur in the saturated ash deposits below Elev. 870 ft. Three cross sections through the new embankment (crest elevation of 962 ft) were considered. In each case, the future ash deposits within the new lined impoundment were assumed to liquefy, with a residual strength of 50 psf. These results showed that a cross section at Sta. 527+80, located on the north side of the ash basin, might exhibit an unacceptable factor of safety against failure if the underlying ash were to liquefy.

Hence, the liquefaction assessment focused on the conditions beneath the existing dike at Sta. 527+80 (Figure 10). Here, the pre-1989 ash deposits extend beneath the full width of the existing dike. Calculations were carried out for the stresses at an elevation of 860 ft (approximate mid-depth of the saturated ash deposits) with an assumed phreatic surface at 870 ft. The factor of safety against liquefaction was computed on 50-ft offsets across the cross section, from the toe of the existing dike to the centerline of the future gypsum embankment (see Section 4.4).

## 4.2. Representative CPT Tip Resistance

Several "Dutch Cone" tests were completed in the area of the critical section (Sta. 527+80) in 1989 (FMSM Engineers 2006). Because the tests were conducted prior to construction of the dike, the 1989 data does not fully represent the current condition of the ash, which has been consolidated by the weight of the dike. Cone penetration tests were not conducted in this area during the 2005 field explorations. However, the 2005 data, which was obtained using an electronic CPT probe and is better documented, is considered more reliable than the 1989 Dutch cone data. Accordingly, the 2005 CPT data were used to assess the liquefaction resistance of the basin deposits, with appropriate adjustments made to account for the stress conditions under the dike.

Fourteen CPT explorations were completed in the ash basin in 2005 (FMSM Engineers 2006). The clean-sand equivalent, corrected, normalized tip resistance values  $[(q_{c1N})_{cs}]$  from each of these tests are plotted in Figure 11. Concentrating on the lower range of the  $(q_{c1N})_{cs}$  data, and ignoring the data peaks, the basin deposits can be characterized by three bands. At the top, the softest deposits are observed above Elev. 870 ft (in CPT-08, CPT-08A, and CPT-9). A slightly stronger stratum is observed below Elev. 870 ft, down to about Elev. 815 ft. The stronger material beneath corresponds to the coarser material deposited prior to 1973.

In the critical section, the liquefaction assessment focused on saturated material beneath the dike, approximately between elevations 850 ft and 870 ft. This corresponds in elevation to the middle band of material penetrated in the basin in the 2005 CPTs. This material was sluiced to the basin in the years between 1973 and 1989. Given the similar age, source, and stress levels, the basin deposits in this depth interval should be similar to those found beneath the dike. While the ash gradation may vary with spatial location in the pond, a reasonable and conservative assumption is that the weaker material penetrated in the basin is representative of the ash deposits beneath the dike.

With this understanding, the data in Figure 11 was re-plotted using expanded scales in Figure 12. Over the depth interval between Elevs. 850 and 870 ft, most of the low-end  $(q_{c1N})_{cs}$  values vary between about 12 and 50. The lower one-third of this range is a  $(q_{c1N})_{cs}$  of about 25. Accordingly, the ash deposits in this depth interval can be characterized using  $(q_{c1N})_{cs}$ =2 (low value) or  $(q_{c1N})_{cs}$ =25 (lower one-third value).

The CPT-based correlation for liquefaction resistance (Equ. 4 and Figure 4) was developed from case histories, where an average cone penetration resistance was usually selected to represent the liquefied soil layer (Robertson and Wride 1998). Hence, application of the criteria to individually measured  $(q_{c1N})_{cs}$  values can be conservative, in that the lowest values of  $(q_{c1N})_{cs}$  in a variable deposit may not be indicative of the overall strength of the layer.

# 4.3. Assumptions and Sources of Uncertainty

# 4.3.1. Cyclic Stress Ratio

The cyclic stress ratio (CSR) was estimated using the simplified procedure (Section 3.1). Significant uncertainty results from the application of an "average" stress reduction factor (r<sub>d</sub>, Equ. 3). More reliable results could be obtained from a ground response analysis.

In addition, CSR values were computed assuming a<sub>max</sub> equal to the PGA values in Table 1; these PGA values actually represent peak bedrock motions. As the seismic stress waves propagate upward through a soil profile, peak accelerations may be amplified or attenuated, depending on the characteristics of the soil. The definition of CSR (Equ. 2) is based on ground surface accelerations; assuming (a<sub>max</sub>)<sub>surface</sub>=PGA<sub>rock</sub> will be *unconservative* if the ground motions are amplified by the soft ash deposits.

## 4.3.2. Correction for Fines Content

The procedures outlined in Sections 3.2.1 and 3.2.2 were followed in computing the values of  $(q_{c1N})_{cs}$  plotted in Figure 11 and Figure 12. For the majority of the CPT data points, the soil behavior type index ( $I_c$ , Equ. 11) was greater than 2.6, where the grain characteristic correction factor ( $K_c$ , Equ. 13 and Figure 6) is considered not applicable (Robertson and Wride 1998). The  $K_c$  correlation is generally limited to silty sands having a fines content (percent finer than 0.075 mm) less than about 35%, whereas the fines content in much of the ash basin deposits exceeds 75 to 90% (FMSM Engineers 2006).

For the liquefaction analysis of the fly ash deposits,  $K_c$  was computed using Equ. 13 with an assumed, limiting maximum value of  $K_c$ =5.0. This maximum value corresponds to the upper limit of the plot in Figure 6 for values of  $I_c$  greater than about 2.825. In developing their correlation (Figure 6) between  $K_c$  and  $I_c$ , Robertson and Wride (1998) considered similar relationships suggested by earlier researchers; the relationship shown in Figure 6 and Equ. 13 was placed conservatively below the other correlations. Robertson and Wride suggested limiting application of the correlation to  $I_c$ <2.6 in the belief that soils with  $I_c$ >2.6 were generally nonliquefiable. However, fly ash is potentially liquefiable-regardless of the  $I_c$  index value indicated by the CPT data. Some of the other relations considered by Robertson and Wride (1998) extend to  $K_c$  values greater than five for  $I_c$ =2.6. Overall, using  $K_c$  values up to a limiting maximum value 5.0 when  $I_c$ >2.6 in fly ash seems reasonable.

To better understand the basis for selecting the limiting value of  $K_c$ , consider three soils having the same CRR: a clean sand, a silty sand, and a nonplastic silt. A silty sand will have a lower penetration resistance than a clean sand at the same cyclic strength. Robertson and Wride (1998) used the  $K_c$  factor to adjust the  $q_{c1N}$  in silty sand upward to a value equivalent to what would be measured in the clean sand. Now consider the nonplastic silt (or fly ash) with the same cyclic strength. The penetration resistance is expected to be lower in the silt than in the clean sand, due to a higher compressibility of the silt. A  $K_c$  factor greater than one is thus appropriate for fly ash. Furthermore, as the silt content of a silty sand increases, the behavior is increasingly controlled by the silt and, at some high silt content, the sand grains will be floating in the finer material and contribute little to the behavior. Hence, assuming a transition in behavior as the silt content increases, the value of  $K_c$  for a nonplastic silt should be more than that for a silty sand. Lacking more definitive data on the differences between  $q_{c1N}$  in clean sand and fly ash at the same CRR, limiting  $K_c$  to a maximum value of 5.0 appears to be appropriate.

## 4.3.3. Representative CPT Values

The softer basin deposits can be characterized using a low value of  $(q_{c1N})_{cs}=12$  or a lower one-third value  $(q_{c1N})_{cs}=25$  (Section 4.2). Both values conservatively represent the weaker ash deposits penetrated in the basin. These values are in the low range for Equ. 4, where small differences in  $(q_{c1N})_{cs}$  do not translate into large differences in CRR<sub>7.5</sub>.

In addition, values of  $(q_{c1N})_{cs}$ =12 to 50 fall within a region of Figure 4 where there is little field data to constrain the curve defining the cyclic strength (CRR). Hence, there is inherent uncertainty regarding the liquefaction resistance of soils exhibiting such low values of penetration resistance.

#### 4.3.4. Correction for Overburden Stress

The correction for overburden stress ( $K_{\sigma}$ , Equ. 15 and 16) depends on the relative density of the soil. The relative density of silt is difficult to measure, and the available CPT-based correlations are generally not valid for silts. However, because the ash deposits are young, were sluiced to the basin, and have not been compacted, the ash deposits were assumed to be loose (low relative density). Accordingly,  $K_{\sigma}$  was computed using a minimum value of f=0.8, corresponding to an assumed relative density less than 40%.

#### 4.3.5. Correction for Static Shear Stress

Assuming the relative density of the ash is less than 40% (Section 4.3.4), a reduction in cyclic resistance is warranted to account for the effects of the static shear stress under the embankment (Section 3.2.5). The magnitude of the correction is not well defined for a variety of soils and stress conditions; the NCEER workshop (Youd et al. 2001) did not reach a consensus recommendation on the specification of  $K_{\alpha}$  values. However, given the apparent significance of this effect at this site, the approximate definition for the  $K_{\alpha}$  correction factor (Equ. 18) was used in the liquefaction assessment.

The static shear stress ratio ( $\alpha$ , Equ. 17) at different locations within a liquefiable soil can be obtained from a numerical model of the embankment. Lacking such an analysis for the design cross section, the value of  $\alpha$  was estimated from the results of a static slope stability analysis. The limit-equilibrium factor of safety for slope stability is defined as:

$$FS_{slope} = \frac{\text{shear strength of soil}}{\text{shear stress required for equilibrium}} = \frac{c' + \sigma' \tan \phi'}{\tau}$$
 19

Under static conditions, the strength of the ash deposits are characterized using c'=0 and  $\phi$ ' =32°. The critical section at Sta. 527+80 was found to have a static FS<sub>slope</sub>=2.2. Substituting and rearranging terms:

$$\frac{\tau}{\sigma'} = \frac{\tan \phi'}{FS_{slope}} = \frac{\tan 32^{\circ}}{2.2} = 0.284$$

20

That is, the ratio of the static shear stress to normal effective stress is 0.284 along the critical slip surface in the ash deposits beneath the embankment. Along all other potential slip surfaces, the factor of safety is higher and this stress ratio is less. For the liquefaction assessment, we need the ratio of the static *horizontal* shear stress to the *vertical* effective stress at the location where the CRR is to be computed. Given these considerations, an approximate value of  $\alpha$ =0.2 was assumed for the liquefaction evaluation. A value of  $\alpha$ =0.2

is consistent with values computed in a finite difference model (accomplished using FLAC 5) of a different dam, which had dimensions and soil properties similar to the embankment cross section at Sta. 527+80.

For  $D_R$ <40% and  $\alpha$ =0.2, the resulting  $K_\alpha$  correction factor is 0.6. Hence, while the correction for static shear stresses is not well defined, the cyclic resistance of the ash was conservatively reduced by 40% to account for this effect.

# 4.3.6. Liquefaction Potential in the Capillary Fringe

A significant capillary fringe of saturated to nearly saturated material is expected to remain for about 20 ft above the phreatic surface (ground water table) in the ash deposits. During a seismic event, significant excess pore pressures may develop in the capillary fringe within the fly ash. Accordingly, the saturated ash is considered susceptible to liquefaction below the phreatic surface and within the capillary fringe (elevations<phreatic surface+20 ft).

Surface tension (capillarity) causes water to rise in a soil column above the phreatic surface or water table (elevation of zero pore water pressure). The height of capillary rise in a soil varies inversely with the radius the pore spaces, which is related to the grain size distribution. In silts, the maximum height of capillary rise ranges up to about 30 ft (Fredlund and Rahardjo 1993). In the ash basin, the height of ash saturated by capillarity was assumed to be 20 ft.

The existing basin deposits are primarily fly ash, with variable mixtures of coarser bottom ash in some locations. Because of the depositional sequence, thin layers of varying gradation exist in the basin. Coarse layers can act as "capillary breaks", which stop the upward migration of a wetting front due to capillarity. However, because the saturated basin deposits will drain and dewater, the top of the capillary fringe will drop in the basin over time (drying front). Under these conditions, coarser layers in the ash will not prevent the formation of a residual capillary fringe to full height in the fly ash.

Suction pressures, which contribute to higher interparticle stresses in a soil, will exist in the capillary fringe and unsaturated zone above the phreatic surface. In the Field Performance Test, suction pressures were measured directly in the ash with tensiometers (FMSM Engineers 2006). Negative (suction or tensile) pore water pressure will pull the ash particles together, thereby increasing the resistance to liquefaction in a seismic event. In the liquefaction assessment, the pore pressure profile was assumed to decrease (become increasingly negative) linearly with height above the phreatic surface up to the top of the assumed capillary fringe (up to 20 ft above the phreatic surface). This assumed pore pressure profile represents an equilibrium condition, with no downward infiltration or upward migration of water in the ash.

More than 20 ft above the phreatic surface, the ash was assumed to be unsaturated and not prone to significant strength loss. The compressibility of the pore air mitigates the development of excess pore pressures and liquefaction in unsaturated, particulate soils. As the saturation of sand is lowered, the cyclic strength increases rapidly (Yang et al. 2004). For example, Ishihara et al. (2001) published data showing that the cyclic strength of one sand roughly doubled as the degree of saturation was reduced from 100% to 90%.

## 4.4. Results

#### 4.4.1. Phreatic Surface at Elev. 870 ft.

The long-term phreatic surface in the basin is expected to be no higher than Elev. 870 ft. For this groundwater level, the factor of safety against liquefaction was evaluated on 50-ft offsets in the cross section at Sta. 527+80 (Figure 10). The evaluations were made at Elev. 860 ft, at mid-depth of the saturated ash beneath the existing dike in this critical section for stability.

The results of the liquefaction calculations are summarized in Figure 13 through Figure 15. Calculations for the local design earthquake (5.00  $M_w$  and PGA =0.098 g), for  $(q_{c1N})_{cs}$  values of 12 and 25, are given in the top and bottom of Figure 13, respectively. Corresponding results for the mean (6.32  $M_w$  and PGA=0.053 g) and modal (7.70  $M_w$  and PGA=0.037 g) design earthquakes are presented in Figure 14 and Figure 15. In each case, the computed factor of safety (FS<sub>lig</sub>) at the 50-ft spacings are indicated in the highlighted rows.

In all cases evaluated with the water table at Elev. 870 ft, liquefaction is predicted to occur at the toe of the existing dike. Under the crest of the new gypsum embankment (off set=+230 ft), the effective stresses are high enough to mitigate liquefaction of the ash deposits for each case considered. The lateral extent of the liquefied soil depends on the severity of the earthquake. Considering the low-end value of  $(q_{c1N})_{cs}$ =12, liquefaction is predicted over a greater width than if the lower one-third value of  $(q_{c1N})_{cs}$ =25 is used.

The dynamic stability of the cross section at Sta. 559+09 was further evaluated using pseudostatic slope stability analyses. Liquefaction of the ash to a residual strength ( $S_{iiq}/\sigma'_{vo}=0.04$ ) was assumed where  $FS_{iiq}<1.1$ . No liquefaction, but with a 20% strength loss ( $\phi'_{red}=27^{\circ}$ ), was assumed where  $FS_{iiq}>1.4$ . In the intermediate zones where 1.1<F $S_{iiq}<1.4$ , the undrained strength of the ash was assumed to be one-third of the static shear strength ( $S_{u}/\sigma'_{v}\approx \frac{1}{3}\tan \phi'\approx 0.2$ ); this intermediate strength, for ash exhibiting high pore pressures and approaching a liquefied state, is simply a guess. Two conditions were analyzed for stability, with differing lateral extents of liquefaction, as summarized in Table 2. In both cases, the factor of safety for slope stability ( $FS_{slope}$ ) is less than one, indicating failure for the assumed conditions.

Table 2. Assessment of Slope Stability at Sta. 527+80 with Liquefaction Evaluated Assuming a Phreatic Surface at Elev. 870 ft.

Design Earthquake Event =	Local Event	Modal Event
Normalized CPT (q <sub>c1N</sub> ) <sub>cs</sub>	12	12
Liquefaction evaluation	Top of Figure 13	Top of Figure 15
Lateral extent liquefied ash (FS <sub>lig</sub> < 1.1 at Elev. 860 ft)	60 ft or more to left Of dike centerline	50 ft or more to left of dike centerline
Lateral extent unliquefied ash (FS <sub>lig</sub> > 1.4 at Elev. 860 ft)	to right of dike centerline	50 ft or more to right of dike centerline
Horizontal pseudostatic coefficient	0.05	0.02
FS <sub>slope</sub> for slope stability (circular failure surface)	0.7	0.6

#### 4.4.2. Lowered Phreatic Surface

The liquefaction and slope stability analyses summarized in Section 4.4.1 indicate an unacceptable performance for the new embankment in the area of the existing north dike. Assuming a phreatic surface at Elev. 870 ft, liquefaction and slope failure are expected in this area for the both the local (5.00 M<sub>w</sub>, PGA=0.098 g) and modal (7.70 M<sub>w</sub>, PGA=0.037 g) design earthquakes.

On the other hand, the phreatic surface is expected to recede to an elevation below 870 ft across the basin. Additional analyses were carried out assuming the ground water table will be at Elev. 856 ft, or one foot above the bottom of the ash in the critical section at Sta. 527+80 (Figure 10). The factor of safety against liquefaction was evaluated at an offset of 90 ft to the left of the dike centerline, in an area where the ash deposits extend beyond the toe of the existing dike. Calculations were made at 1-ft spacings between the top and bottom of the ash deposit (Elev. 864 to 855 ft) at this location. In each case, the vertical effective stress was computed assuming a linear increase in negative pore pressure above the phreatic surface (Section 4.3.6).

The results of this analysis, for  $(q_{c1N})_{cs}$ =12 or 25, are summarized in Figure 16 through Figure 18 for the three design earthquake scenarios. Examination of the results shows  $FS_{liq} \ge 1.4$  throughout (the lowest value is  $FS_{liq} = 1.39$  at the bottom of the ash, for the modal earthquake and  $(q_{c1N})_{cs} = 12$ ). At vertical profiles at other offsets in this cross section,  $FS_{liq}$  will be greater due to the higher vertical stresses under the embankments.

The results from stability analyses of the embankments with a phreatic surface at Elev. 856 ft are summarized in Table 3. Liquefaction is not predicted under these conditions, so the strength of the saturated ash was reduced only 20% at all locations below Elev. 876 ft (phreatic surface+20 ft). The stability analyses indicate adequate factors of safety under these conditions.

Table 3. Assessment of Slope Stability at Sta. 559+09 with Liquefaction Evaluated Assuming a Phreatic Surface at Elev. 870 ft.

Design Earthquake Event =	Local Event	Modal Event
Normalized CPT (q <sub>c1N</sub> ) <sub>CS</sub>	12	12
Liquefaction evaluation	Top of Figure 16	Top of Figure 18
Lateral extent liquefied ash	No liquefaction	No liquefaction
Horizontal pseudostatic coefficient	0.05	0.02
FS <sub>slope</sub> for slope stability (circular failure surface)	1.8	2.0

## 4.4.3. Conclusion

A critical location for potential liquefaction in the expanded ash basin facility occurs at Sta. 527+80. Here, the pre-1989 ash deposits extend beneath the full width of the existing dike. Based on CPT penetrations in the pond, the ash in the vicinity of the slope toe (offsets of 80 to 100 from the dike centerline) is predicted to liquefy if the phreatic surface is at Elev. 870 ft during any of the three earthquake scenarios evaluated. Liquefaction in this area will destabilize the existing dike, will likely cause progressive sliding of the larger embankment, and may lead to the loss of containment.

The assumed phreatic surface elevation of 870 ft is somewhat conservative. The location of the long-term phreatic surface can not be determined at this time, as water from the existing basin controls the ground water hydraulics in this vicinity. After the inflows are permanently diverted from the basin at the start of construction, the phreatic surface is expected to gradually drain to a level below Elev. 870 ft. The analysis in Section 4.4.2 indicates that liquefaction will not occur, and the embankments will be stable, during the three design earthquake events if the phreatic surface is at or below Elev. 856 ft.

The project design should therefore include a plan for monitoring groundwater levels beneath the dike along the north end of the basin, and in the area between the dike and the railroad tracks. If the measured ground water table does not drop to an elevation of 856 ft or lower, then mitigation measures should be undertaken to lower the water levels or otherwise stabilize the embankment against a potential liquefaction failure.

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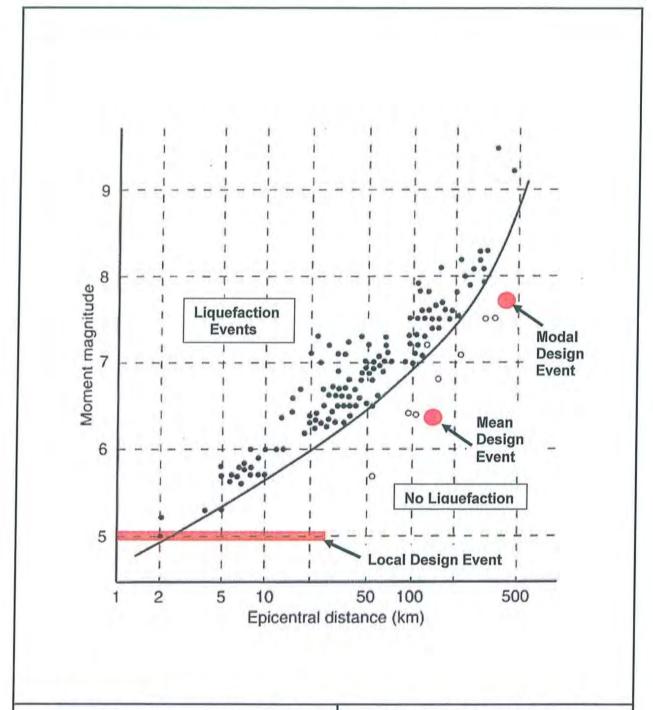
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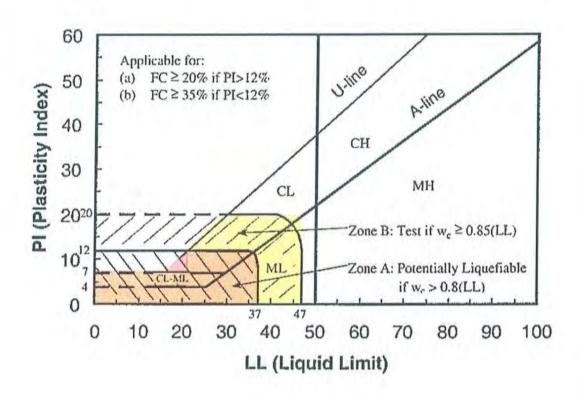


E. W. Brown Generating Station Main Ash Pond Design Mercer County, Kentucky





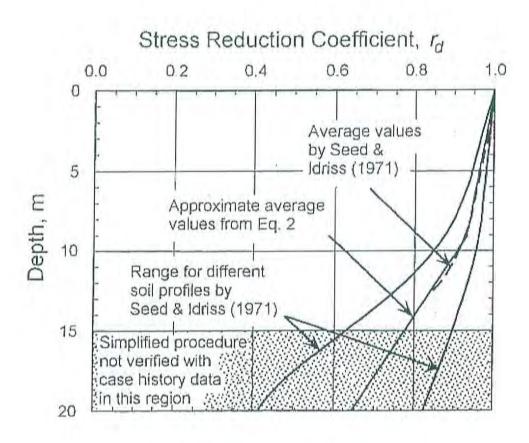
Figure 1. Maximum Epicentral
Distance to Observed
Liquefaction Events in
Shallow (Focal Depth < 50
km) Earthquakes (from
Kramer 1996, after
Ambraseys 1988)



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Figure 2. Criteria for Assessing Liquefaction in Fine Grained Soils (after Seed et al. 2003).

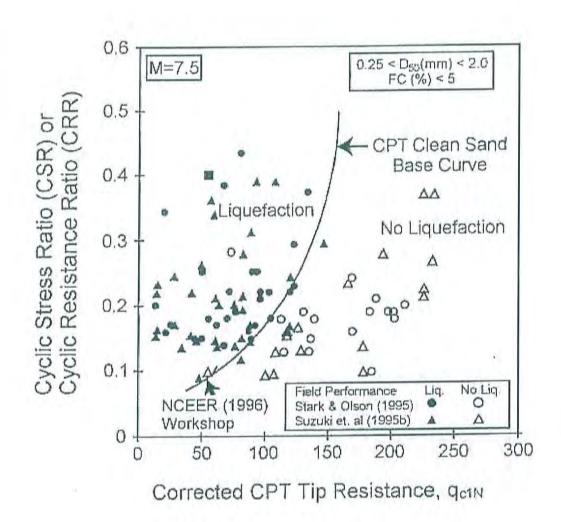


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Figure 3. Variation of the Stress Reduction Factor with Depth (from Youd et al. 2001).

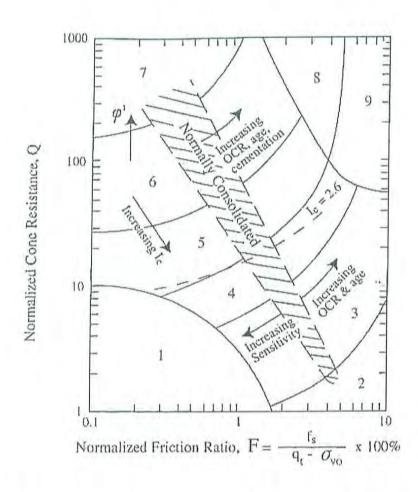


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Figure 4. Curve and Case History Data Relating Cone Penetration Resistance to Liquefaction Resistance (from Youd et al. 2001, after Robertson and Wride 1998).

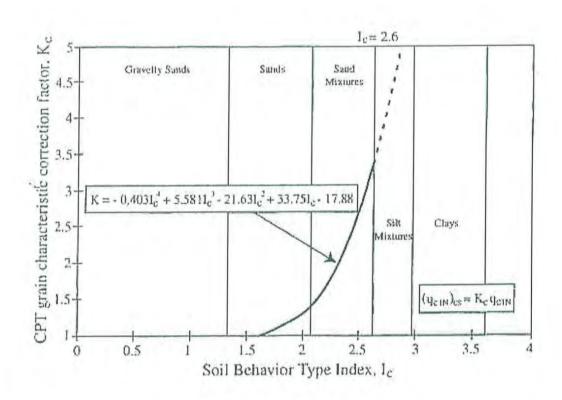


- L. Sensitive, fine grained
- 2. Organic soils peats
- 3. Clays silty clay to clay
- 4. Silt mixtures clayey silt to silty clay
- 5. Sand mixtures silty sand to sandy silt
- 6. Sands clean sand to silty sand
- 7. Gravelly sand to dense sand
- 8. Very stiff sand to clayey sand\*
- 9. Very stiff, fine grained\*

\*Heavily overconsolidated or cemented

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Soil Behavior-Type Chart for Figure 5. Classifying Soils from CPT Data (from Youd et al. 2001, after Robertson 1990).

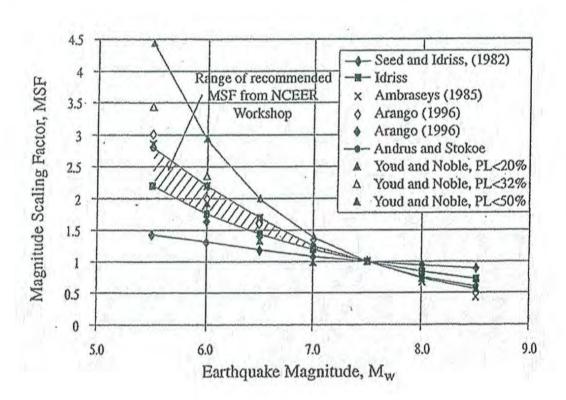


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Figure 6.

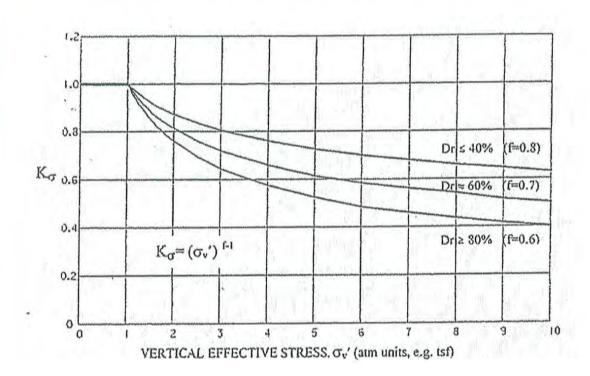
Grain Size Correction Factor (K<sub>c</sub>) for Determination of a Clean-Sand Equivalent CPT Tip Resistance (from Youd et al. 2001, after Robertson and Wride 1998).



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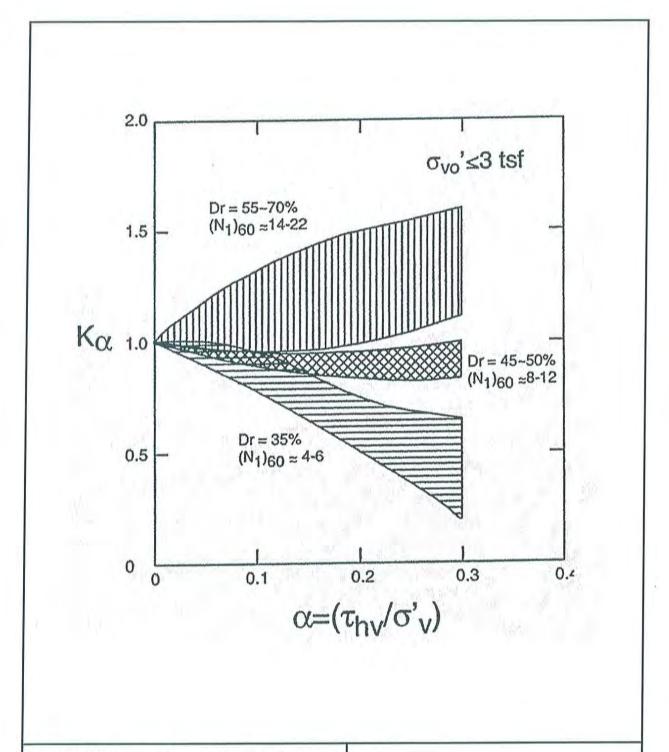
Figure 7. MSF for Correcting CRR for Earthquake Magnitude (from Youd et al. 2001)



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Figure 8. K<sub>o</sub> Factor for Correcting CRR for Overburden Pressure (from Youd et al. 2001).

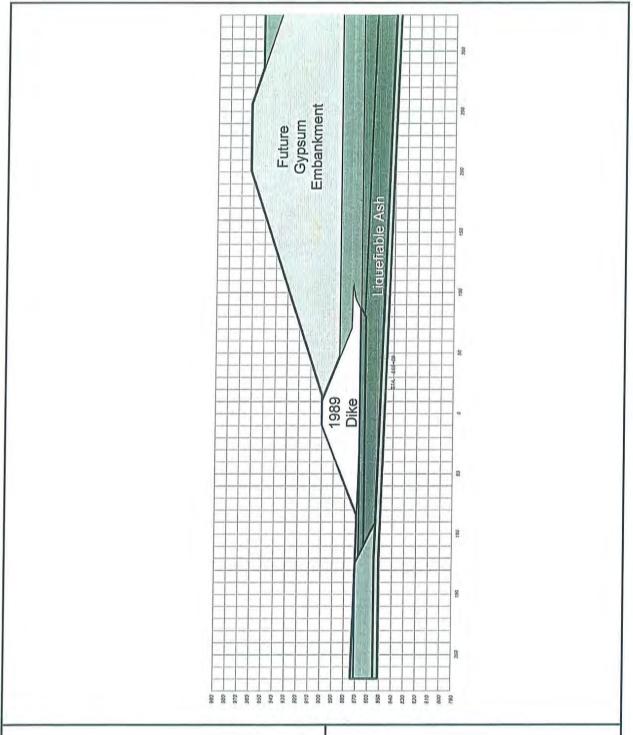


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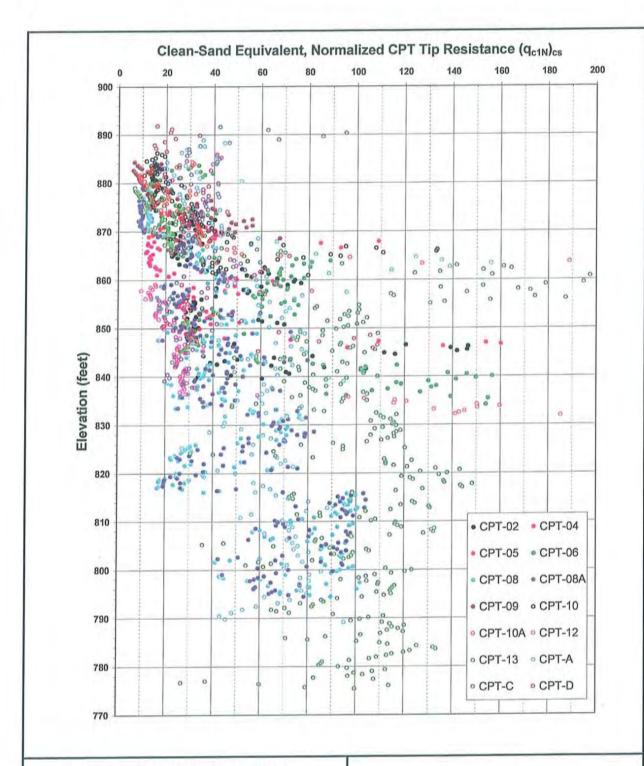
Figure 9. K<sub>α</sub> Factor for Correcting CRR for Static Driving Shear Stresses (from Seed et al. 2003, after Harder and Boulanger 1997)



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Figure 10. Critical Cross Section (Sta. 527+80) for Assessment of Liquefaction Potential.



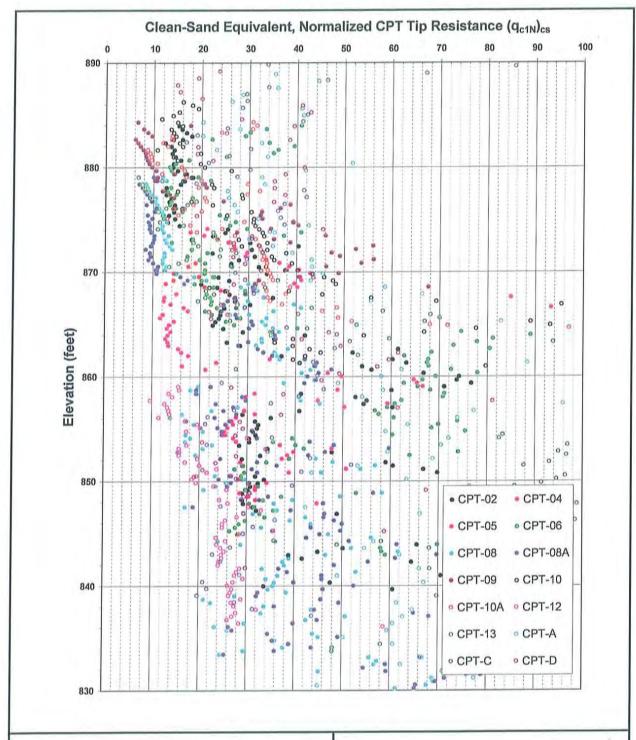
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Liquefaction All CPT Plot.xls

Figure 11. Corrected, Normalized CPT
Tip Resistance Measured in
the Ash Basin Deposits.



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Figure 12. Corrected, Normalized CPT
Tip Resistance Measured in
the Ash Basin Deposits
(Expanded Scales).

Representative normalize  Design earthqua  Earthquake m  Peak ground acceleration	ake event =	LOCAL	c1N)_cs =	12 Phres	atic surface e			
Earthquake m	nagnitude =			Phres				
Earthquake m	nagnitude =					870 ft 118 pcf		
					Unit weight of embankment =			f
Teak ground additional		0.098 g			nit weight of		100 pc	
	II dii iddii -	9,549, 9			iir ii aigi ii -ii -ii		10000	
Offset from dike centerline =	-100	-50	0	50	100	150	200	250
Elevation of ground surface ~	871	884	900	913	929	945	961	962 (
Elevation at base of embankment ~	864	864	864	864	885	885	885	885 (
Elevation for assessment =	860	860	860	860	860	860	860	860 (
Depth below surface =	11	24	40	53	69	85	101	102 f
Depth below surface =	3.4	7.3	12.2	16.2	21.0	25.9	30.8	31.1 r
Assumed surface PGA =	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
Stress reduction factor (r_d) =	0.974	0.944	0.848	0.743	0.612	0.560	0.560	0.560
Total vertical stress =	1226	2760	4648	6182	7692	9580	11468	11586 p
Static pore water pressure =	624	624	624	624	624	624	624	624 p
Effective vertical stress =	602	2136	4024	5558	7068	8956	10844	10962 p
Cyclic Stress Ratio (CSR) =	0.126	0.078	0.062	0.053	0.042	0.038	0.038	0.038
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
Magnitude scaling factor (MSF) =	2.82	2.82	2.82	2.82	2.82	2.82	2.82	2.82
Overburden stress correction factor (K_sigma) =	1.00	1.00	0.88	0.82	0.79	0.75	0.72	0.72
Static shear stress correction factor (K_alpha) =	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Cyclic Resistance Ratio (CRR) =	0.102	0.101	0.089	0.084	0.080	0.076	0.073	0.073
Factor of safety against ilquefaction (FS_liq) =	0.80	1,31	1,43	1.59	1.88	2,00	1,94	1,94
Representative normalize	d CPT tip res	sistance (q_c	:1N)_cs =	25				
Design earthqual	ke event =	LOCAL		Phreat	tic surface el	evation =	870 ft	
Earthquake ma	agnitude =	5.00 M	w		ight of emba		118 pc	
Peak ground acceleration	on rock =	0.098 g		Uni	it weight of c	oal ash =	100 pc	
Office there allow a sectoral as a	400	50		50	100	150	200	250 f
Offset from dike centerline =	-100 871	-50 884	900	913	929	945	961	962 ft
Elevation of ground surface ~ Elevation at base of embankment ~	864	864	864	864	885	885	885	885 ft
	860	860	860	860	860	860	860	860 ft
Elevation for assessment = Depth below surface =	11	24	40	53	69	85	101	102 ft
Depth below surface =	3.4	7.3	12.2	16.2	21.0	25.9	30.8	31.1 n
Assumed surface PGA =	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098 g
Stress reduction factor (r_d) =	0.098	0.944	0.848	0.743	0.612	0.560	0.560	0.560
Total vertical stress =	1226	2760	4648	6182	7692	9580	11468	11586 p
Static pore water pressure =	624	624	624	624	624	624	624	624 p
Effective vertical stress =	602	2136	4024	5558	7068	8956	10844	10962 p
	0.126	0.078	0.062	0.053	0.042	0.038	0.038	0.038
Cyclic Stress Ratio (CSR) =					E.Carl	222	0.074	0.074
Cyclic Stress Ratio (CSR) =	0.071	0.071	0.071	0.071	0.071	0.071	0.071	0.071
Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	0.071 2.82	0.071	0.071 2.82	0.071 2.82	2,82	0.071 2.82	2.82	2.82
Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =  Magnitude scaling factor (MSF) =								
Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =  Magnitude scaling factor (MSF) =  Overburden stress correction factor (K_sigma) =	2.82	2.82	2.82	2.82	2.82	2.82	2.82	2.82 0.72 0.6
Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	2.82 1.00	2.82 1.00	2.82 0.88	2.82 0.82	2.82 0.79	2.82 0.75	2.82 0.72	2.82 0.72

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Liquefaction Assessment.xls

Figure 13. Liquefaction
Assessment in the
Section at Sta.
527+80 for the Local
Design Earthquake.

Representative normalize	d CPT tip re	sistance (q_c	:1N)_cs =	12				
Design earthqual	ce event =	MEAN			tic surface e		870 ft	
Earthquake ma	ignitude =	6.32 M	W		ight of emba		118 pc	
Peak ground acceleration	on rock =	0.053 g		Un	it weight of c	coal ash =	100 pc	of
				100	765	500		
Offset from dike centerline =	-100	-50	0	50	100	150	200	250 f
Elevation of ground surface ~	871	884	900	913	929	945	961	962 ft
Elevation at base of embankment ~	864	864	864	864	885	885	885	885 ft
Elevation for assessment =	860	860	860	860	860	860	860	860 11
Depth below surface =	11	24	40	53	69	85	101	102 ft
Depth below surface =	3.4	7.3	12.2	16.2	21.0	25.9	30.8	31.1 m
Assumed surface PGA =	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053 g
Stress reduction factor (r_d) =	0.974	0.944	0.848	0.743	0.612	0.560	0,560	0.560
Total vertical stress =	1226	2760	4648	6182	7692	9580	11468	11586 p
Static pore water pressure =	624	624	624	624	624	624	624	624 p
Effective vertical stress =	602	2136	4024	5558	7068	8956	10844	10962 p
Cyclic Stress Ratio (CSR) =	0.068	0.042	0.034	0.028	0.023	0.021	0.020	0.020
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
Magnitude scaling factor (MSF) =	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55
Overburden stress correction factor (K_sigma) =	1.00	1.00	0.88	0.82	0.79	0.75	0.72	0.72
Static shear stress correction factor (K_alpha) =	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Cyclic Resistance Ratio (CRR) =	0.056	0.056	0.049	0.046	0.044	0.042	0.040	0.040
Factor of safety against liquefaction (FS_liq) =	0.82	1.32	1.45	1.62	1.91	2.03	1.97	1.97
Representative normalized Design earthquak Earthquake ma	e event =	MEAN 6,32 M			ic surface el		870 ft 118 pc	1
Peak ground acceleration		0.053 g	9	Uni	t weight of c	oal ash =	100 pc	1
Offset from dike centerline =	-100	-50	0	50	100	150	200	250 ft
Elevation of ground surface ~	871	884	900	913	929	945	961	962 ft
Elevation at base of embankment ~	864	864	864	864	885	885	885	885 ft
Elevation for assessment =	860	860	860	860	860	860	860	860 ft
Depth below surface =	11	24	40	53	69	85	101	102 ft
Depth below surface =	3.4	7.3	12.2	16.2	21.0	25.9	30.8	31.1 m
Assumed surface PGA =	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053 g
Stress reduction factor (r_d) =	0.974	0.944	0.848	0.743	0.612	0.560	0.560	0.560
Total vertical stress =	1226	2760	4648	6182	7692	9580	11468	11586 pt
Static pore water pressure =	624	624	624	624	624	624	624	624 ps
Effective vertical stress =	602	2136	4024	5558	7068	8956	10844	10962 ps
Cyclic Stress Ratio (CSR) =	0.068	0.042	0.034	0.028	0.023	0.021	0.020	0.020
a Yalla Billana Cianta (A. L. IV.		0.071	0.071	0.071	0.071	0.071	0.071	0.071
	0.071		7 1 7 1		4 00	4 22	1.55	1.55
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	0.071 1.55		1.55	1.55	1.55	1.55	1,00	
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =  Magnitude scaling factor (MSF) =	0.071 1.55 1.00	1.55	1.55 0.88	1.55 0.82	0,79	0.75	0.72	0.72
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =  Magnitude scaling factor (MSF) =  Overburden stress correction factor (K_sigma) =	1.55	1.55					0.72 0.6	0.72
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	1.55 1.00	1.55 1.00	0.88	0.82	0.79	0.75	0.72	0.72
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) = Overburden stress correction factor (K_sigma) = Static shear stress correction factor (K_sipha) =	1.55 1.00 0.6	1.55 1.00 0.6	0.88	0.82	0.79	0.75	0.72 0.6	0.72

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Liquefaction Assessment.xis

Figure 14. Liquefaction
Assessment in the
Section at Sta.
527+80 for the Mean
Design Earthquake.

Topi obalismy i istii diiko	d CPT tip re	sistance (q_c	c1N)_cs =	12				
Design earthquak Earthquake ma	ignitude =	MODAL 7.70 M	lw	Unit we	tic surface e	inkment =	870 ft 118 pc 100 pc	of
Peak ground acceleration	on rock =	0.037 g		Un	it weight of d	coai asn =	100 pc	4
AND THE COURT OF T	-100	-50	0	50	100	150	200	250 ft
Offset from dike centerline =	871	884	900	913	929	945	961	962 ft
Elevation of ground surface ~ Elevation at base of embankment ~	B64	864	864	864	885	885	885	885 ft
Elevation for assessment =	860	860	860	860	860	860	860	860 ft
Depth below surface =	11	24	40	53	69	85	101	102 ft
Depth below surface =	3.4	7.3	12.2	16.2	21.0	25.9	30.8	31.1 m
Assumed surface PGA =	0.037	0.037	0.037	0.037	0.037	0.037	0.037	0.037 g
	0.974	0.944	0.848	0.743	0,612	0.560	0.560	0.560
Stress reduction factor (r_d) =	1226	2760	4648	6182	7692	9580	11468	11586 p
Total vertical stress =			624	624	624	624	624	624 p
Static pore water pressure =	624	624		5558	7068	8956	10844	10962 p
Effective vertical stress =	602	2136	4024			0.014	0.014	0.014
Cyclic Stress Ratio (CSR) =	0.048	0.029	0.024	0.020	0.016	0.014		
Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	0.060	0.060	0.060	0.060	0.060	0,060	0.060	0.060
Magnitude scaling factor (MSF) =	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Overburden stress correction factor (K_sigma) =	1.00	1.00	0.88	0.82	0.79	0.75	0.72	0.72
Static shear stress correction factor (K_alpha) =	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Cyclic Resistance Ratio (CRR) =	0.034	0.034	0.030	0.028	0.026	0.025	0.024	0.024
Factor of safety against liquefaction (FS_liq) =	0.70	1.14	1.26	1.40	1.65	1.75	1.70	1.70
Representative normalized Design earthquak Earthquake ma Peak ground acceleration	e event = gnitude =	MODAL 7.70 M 0.037 g		Unit wei	ic surface el ght of emba t weight of c	nkment =	870 ft 118 pc 100 pc	
Carry Washington								
	-100	-50	0	50	100	150	200	250 ft
Offset from dike centerline ≒	-100 871		900	50 913	100 929	150 945	200 961	250 ft 962 ft
	-100 871 864	-50 884 864					17.7	
Offset from dike centerline ≒ Elevation of ground surface ~	871	884	900	913	929	945 885 860	961 885 860	962 ft 885 ft 860 ft
Offset from dike centerline ≃ Elevation of ground surface ∼ Elevation at base of embankment ∼ Elevation for assessment ≃	871 864 860	884 864	900 864	913 864	929 885	945 885	961 885	962 ft 885 ft
Offset from dike centerline ⊨ Elevation of ground surface ∼ Elevation at base of embankment ∼	871 864	884 864 860	900 864 860	913 864 860	929 885 860	945 885 860	961 885 860	962 ft 885 ft 860 ft 102 ft
Offset from dike centerline = Elevation of ground surface - Elevation at base of embankment - Elevation for assessment = Depth below surface = Depth below surface =	871 864 860 11 3.4	884 864 860 24	900 864 860 40	913 864 860 53	929 885 860 69	945 885 860 85	961 885 860 101	962 ft 885 ft 860 ft 102 ft 31.1 m
Offset from dike centerline = Elevation of ground surface ~ Elevation at base of embankment ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA =	871 864 860 11 3.4 0,037	884 864 860 24 7.3	900 864 860 40 12.2 0.037	913 864 860 53 16.2	929 885 860 69 21.0	945 885 860 85 25.9	961 885 860 101 30.8	962 ft 885 ft 860 ft 102 ft 31.1 m
Offset from dike centerline = Elevation of ground surface ~ Elevation at base of embankment ~  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) =	871 864 860 11 3.4 0.037 0.974	884 864 860 24 7.3 0.037 0.944	900 864 860 40 12.2 0.037 0.848	913 864 860 53 16.2	929 885 860 69 21.0	945 885 860 85 25.9	961 885 860 101 30.8	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560
Offset from dike centerline = Elevation of ground surface ~ Elevation at base of embankment ~  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress =	871 864 860 11 3.4 0.037 0.974 1226	884 864 860 24 7.3 0.037 0.944 2760	900 864 860 40 12.2 0.037 0.848 4648	913 864 860 53 16.2 0.037 0,743	929 885 860 69 21.0 0.037 0.612	945 885 860 85 25.9 0.037 0.560	961 885 860 101 30.8 0.037 0.560	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 ps
Offset from dike centerline = Elevation of ground surface - Elevation at base of embankment -  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure =	871 864 860 11 3.4 0.037 0.974 1226 624	884 860 24 7.3 0.037 0.944 2760 624	900 864 860 40 12.2 0.037 0.848 4648 624	913 864 860 53 16.2 0,037 0,743 6182 624	929 885 860 69 21.0 0.037 0.612 7692	945 885 860 85 25.9 0.037 0.560 9580	961 885 860 101 30.8 0.037 0.560 11468	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 pt 624 pt
Offset from dike centerline = Elevation of ground surface ~ Elevation at base of embankment ~  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress =	871 864 860 11 3.4 0.037 0.974 1226	884 864 860 24 7.3 0.037 0.944 2760	900 864 860 40 12.2 0.037 0.848 4648	913 864 860 53 16.2 0.037 0,743 6182	929 885 860 69 21.0 0.037 0.612 7692 624	945 885 860 85 25.9 0.037 0.560 9580 624	961 885 860 101 30.8 0.037 0.560 11468 624	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 pt 624 pt
Offset from dike centerline = Elevation of ground surface ~ Elevation at base of embankment ~  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =	871 864 860 11 3.4 0.037 0.974 1226 624 602 0.048	884 860 24 7.3 0.037 0.944 2760 624 2136 0.029	900 864 860 40 12.2 0.037 0.848 4648 624 4024	913 864 860 53 16.2 0.037 0,743 6182 624 5558	929 885 860 69 21.0 0.037 0.612 7692 624 7068	945 885 860 85 25.9 0.037 0.560 9580 624 8956	961 885 860 101 30.8 0.037 0.560 11468 624 10844	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 pt 624 pt 10962 pt 0.014 0.071
Offset from dike centerline = Elevation of ground surface - Elevation at base of embankment -  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) =	871 864 860 11 3.4 0.037 0.974 1226 624 602 0.048	884 860 24 7.3 0.037 0.944 2760 624 2136 0.029	900 864 860 40 12.2 0.037 0.848 4648 624 4024 0.024	913 864 860 53 16.2 0.037 0,743 6182 624 5558 0.020	929 885 860 69 21.0 0.037 0.612 7692 624 7068 0.016	945 885 860 85 25.9 0.037 0.560 9580 624 8956 0.014	961 885 860 101 30.8 0.037 0.560 11468 624 10844 0.014	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 pt 624 pt 10962 pt 0.014
Offset from dike centerline = Elevation of ground surface - Elevation at base of embankment -  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) =	871 864 860 11 3.4 0.037 0.974 1226 624 602 0.048 0.071 0.93	884 860 24 7.3 0.037 0.944 2760 624 2136 0.029 0.071 0.93	900 864 860 40 12.2 0.037 0.848 4648 624 4024 0.024 0.071 0.93	913 864 860 53 16.2 0.037 0.743 6182 624 5558 0.020 0.071 0.93	929 885 860 69 21.0 0.037 0.612 7692 624 7068 0.016 0.071 0.93	945 885 860 85 25.9 0.037 0.560 9580 624 8956 0.014	961 885 860 101 30.8 0.037 0.560 11468 624 10844 0.014	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 pt 624 pt 10962 pt 0.014
Offset from dike centerline = Elevation of ground surface — Elevation at base of embankment —  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) = Overburden stress correction factor (K_sigma) =	871 864 860 11 3.4 0.037 0.974 1226 624 602 0.048 0.071 0.93 1.00	884 860 24 7.3 0.037 0.944 2760 624 2136 0.029 0.071 0.93 1.00	900 864 860 40 12.2 0.037 0.848 4648 624 4024 0.024 0.071 0.93 0.88	913 864 860 53 16.2 0.037 0.743 6182 624 5558 0.020 0.071 0.93 0.82	929 885 860 69 21.0 0.037 0.612 7692 624 7068 0.016 0.071 0.93 0.79	945 885 860 85 25.9 0.037 0.560 9580 624 8956 0.014 0.071 0.93 0.75	961 885 860 101 30.8 0.037 0.560 11468 624 10844 0.014 0.071 0.93 0.72	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 pt 624 pt 10962 pt 0.014 0.071 0.93
Offset from dike centerline = Elevation of ground surface ~ Elevation at base of embankment ~  Elevation for assessment = Depth below surface = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) =	871 864 860 11 3.4 0.037 0.974 1226 624 602 0.048 0.071 0.93	884 860 24 7.3 0.037 0.944 2760 624 2136 0.029 0.071 0.93	900 864 860 40 12.2 0.037 0.848 4648 624 4024 0.024 0.071 0.93	913 864 860 53 16.2 0.037 0.743 6182 624 5558 0.020 0.071 0.93	929 885 860 69 21.0 0.037 0.612 7692 624 7068 0.016 0.071 0.93	945 885 860 85 25.9 0.037 0.560 9580 624 8956 0.014 0.071 0.93	961 885 860 101 30.8 0.037 0.560 11468 624 10844 0.014 0.071	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 ps 624 ps 10962 ps 0.014 0.071 0.93 0.72
Offset from dike centerline = Elevation of ground surface ~ Elevation at base of embankment ~  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  Cyclic Resistance Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) = Overburden stress correction factor (K_sigma) = Static shear stress correction factor (K_sigma) =	871 864 860 111 3.4 0.037 0.974 1226 624 602 0.048 0.071 0.93 1.00 0.6	884 860 24 7.3 0.037 0.944 2760 624 2136 0.029 0.071 0.93 1.00 0.6	900 864 860 40 12.2 0.037 0.848 4648 624 4024 0.024 0.071 0.93 0.88 0.6	913 864 860 53 16.2 0.037 0,743 6182 624 5558 0.020 0.071 0.93 0.82 0.6	929 885 860 69 21.0 0.037 0.612 7692 624 7068 0.016 0.071 0.93 0.79 0.6	945 885 860 85 25.9 0.037 0.560 9580 624 8956 0.014 0.071 0.93 0.75 0.6	961 885 860 101 30.8 0.037 0.560 11468 624 10844 0.014 0.071 0.93 0.72 0.6	962 ft 885 ft 860 ft 102 ft 31.1 m 0.037 g 0.560 11586 ps 624 ps 10962 ps 0.014 0.071 0.93 0.72 0.6

Fuller Mossbarger Scott & May



Liquefaction Assessment.xls

Figure 15. Liquefaction
Assessment in the
Section at Sta.
527+80 for the
Modal Design
Earthquake.

Design earthquak	e event =	LOCAL			ic surface e		856 ft			
Earthquake ma		5.00 N	/w		ght of emba		118 p	cf		
Peak ground acceleration		0.098 g		Uni	t weight of d	coal ash =	100 p	cf		
17.27 A. C.		1000								
Offset from dike centerline =	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90 f
Elevation of ground surface ~	870	870	870	870	870	870	870	870	870	870 f
Elevation at top of ash ~	864	864	864	864	864	864	864	864	864	864 f
Elevation for assessment =	864	863	862	861	860	859	858	857	856	855 f
Depth below surface =	6	7	8	9	10	11	12	13	14	15 f
Depth below surface =	1.8	2.1	2.4	2.7	3,0	3.4	3.7	4.0	4.3	4.6 r
Assumed surface PGA =	0.098	0.098	0.098	0.098	0.098	0.098	0,098	0.098	0.098	0.098
Stress reduction factor (r_d) =	0.986	0.984	0.981	0.979	0.977	0.974	0.972	0.970	0.967	0.965
Total vertical stress =	708	808	908	1008	1108	1208	1308	1408	1508	1608 p
Static pore water pressure =	-499	-437	-374	-312	-250	-187	-125	-62	0	62 p
Effective vertical stress =	1207	1245	1282	1320	1358	1395	1433	1470	1508	1546 p
Cyclic Stress Ratio (CSR) =	0.037	0.041	0.044	0.048	0.051	0.054	0.057	0.059	0.062	0.064
ince Ratio for 7.5 Mw (CRR_7.5) =	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
Magnitude scaling factor (MSF) =	2.82	2.82	2.82	2.82	2.82	2.82	2.82	2.82	2.82	2.82
ress correction factor (K_sigma) =	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
tress correction factor (K_alpha) =	0.6	0.6	0.6	0.6	0,6	0.6	0.6	0.6	0.6	0.6
Cyclic Resistance Ratio (CRR) =	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102
fety against liquefaction (FS_liq) =	2.76	2.50	2.30	2.13	2.00	1.89	1.80	1.72	1,65	1.59
Representative normalized C	CPT tip resi	stance (q_c	1N)_cs =	25						
Design earthquake	e event =	LOCAL		Phreati	surface el	evation =	856 ft			
Earthquake mag	gnitude =	LOCAL 5.00 M	w		surface el ht of embar		118 pc			
	gnitude =		w	Unit weig		nkment =				
Earthquake mag Peak ground acceleration o	gnitude = on rock =	5.00 M 0.098 g		Unit weig Unit	ht of embar weight of co	nkment = oal ash =	118 po 100 po	of .	-00	.pn #
Earthquake mag Peak ground acceleration of Offset from dike centerline =	gnitude = on rock =	5,00 M 0,098 g -90	-90	Unit weig Unit -90	ht of embar weight of co	nkment = pal ash = -90	118 pc 100 pc	ef -90	-90	
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~	gnitude = on rock = -90 870	5,00 M 0,098 g -90 870	-90 870	Unit weig Unit -90 870	ht of embar weight of co -90 870	nkment = oal ash = -90 870	118 pc 100 pc -90 870	-90 870	870	870 ft
Earthquake mag Peak ground acceleration of Offset from dike centerline =	gnitude = on rock =	5,00 M 0,098 g -90	-90	Unit weig Unit -90	ht of embar weight of co	nkment = pal ash = -90	118 pc 100 pc	-90 870 864	870 864	870 ft 864 ft
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~	gnitude = on rock = -90 870	5,00 M 0,098 g -90 870	-90 870	Unit weig Unit -90 870	ht of embar weight of co -90 870	nkment = oal ash = -90 870	118 pc 100 pc -90 870 864 858	-90 870 864	870 864 856	870 ft 864 ft 855 ft
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~	-90 870 864	5,00 M 0,098 g -90 870 864	-90 870 864	Unit weig Unit -90 870 864 861	-90 870 864 860	-90 870 864 859	-90 870 864 858	-90 870 864 857	870 864 856 14	870 ft 864 ft 855 ft 15 ft
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment =	-90 870 864	5,00 M 0,098 g -90 870 864 863	-90 870 864	Unit weig Unit -90 870 864 861	ht of embar weight of co -90 870 864 860	-90 870 864	118 pc 100 pc -90 870 864 858	-90 870 864	870 864 856	870 ft 864 ft 855 ft 15 ft
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface =	-90 870 864 864	5,00 M 0,098 g -90 870 864 863 7	-90 870 864 862 8	Unit weig Unit -90 870 864 861	-90 870 864 860	-90 870 864 859	-90 870 864 858	-90 870 864 857	870 864 856 14	870 ft 864 ft 855 ft 15 ft 4.6 m
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA =	-90 870 864 864 6	5,00 M 0,098 g -90 870 864 863 7 2.1	-90 870 864 862 8	Unit weig Unit -90 870 864 861 9 2.7	-90 870 864 860 10 3.0	-90 870 864 859 11 3.4	-90 870 864 858 12 3.7	-90 870 864 857 13 4.0	870 864 856 14 4.3	870 ft 864 ft 855 ft 15 ft 4.6 m
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) =	-90 870 864 864 6 1.8	5,00 M 0,098 g -90 870 864 863 7 2.1	-90 870 864 862 8 2.4	Unit weig Unit -90 870 864 861 9 2.7	-90 870 864 860 10 3.0	-90 870 864 859 11 3.4	-90 870 864 858 12 3.7	-90 870 864 857 13 4.0	870 864 856 14 4.3	870 ft 864 ft 855 ft 15 ft 4.6 m 0.098 g 0.965 1608 pt
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA =	-90 870 864 864 6 1.8 0.098	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984	-90 870 864 862 8 2.4 0.098 0.981	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979	-90 870 864 860 10 3.0 0.098 0.977	-90 870 864 859 11 3.4 0.098 0.974	-90 870 864 858 12 3.7 0.098 0.972	-90 870 864 857 13 4.0 0.098 0.970	870 864 856 14 4.3 0.098 0.967	-90 ft 870 ft 864 ft 855 ft 4.6 m 0.098 g 0.965 1608 p; 62 p;
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure =	-90 870 864 864 6 1.8 0.098 0.986 708	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984 808	-90 870 864 862 8 2.4 0.098 0.981	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979 1008	-90 870 864 860 10 3.0 0.098 0.977	-90 870 864 859 11 3.4 0.098 0.974 1208	-90 870 864 858 12 3.7 0.098 0.972 1308	-90 870 864 857 13 4.0 0.098 0.970 1408	870 864 856 14 4.3 0.098 0.967 1508	870 ft 864 ft 855 ft 15 ft 4.6 m 0.098 g 0.965 1608 pt 62 pt
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress =	90 870 864 864 6 1.8 0.986 708 -499	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984 808 -437	-90 870 864 862 8 2.4 0.098 0.981 908 -374	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979 1008 -312	-90 870 864 860 10 3.0 0.098 0.977 1108 -250	-90 870 864 859 11 3.4 0.098 0.974 1208 -187	118 pc 100 pc 870 864 858 12 3.7 0.098 0.972 1308 -125	-90 870 864 857 13 4.0 0.098 0.970 1408 -62	870 864 856 14 4.3 0.098 0.967 1508	870 ft 864 ft 855 ft 15 ft 4.6 m 0.098 g 0.965 1608 p 62 p
Earthquake mag Peak ground acceleration of Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress =	90 870 864 864 6 1.8 0.998 0.998 708 708 708 708 708 708 708 708 708 70	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984 808 -437 1245	-90 870 864 862 8 2.4 0.098 0.981 908 -374 1282	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979 1008 -312 1320	-90 870 864 860 10 3.0 0.098 0.977 1108 -250 1358	-90 870 864 859 11 3.4 0.098 0.974 1208 -187 1395	118 pc 100 pc 870 864 858 12 3.7 0.098 0.972 1308 -125 1433 0.057	-90 870 864 857 13 4.0 0.098 0.970 1408 -62 1470 0.059	870 864 856 14 4.3 0.098 0.967 1508 0 1508 0.062	870 ft 864 ft 855 ft 4.6 m 0.098 g 0.965 1608 p: 62 p: 1546 p: 0.064 0.071
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface =  Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =	90 870 864 864 6 1.8 0.986 708 -499 1207 0.037	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984 808 -437 1245 0,041	-90 870 864 862 8 2.4 0.098 0.981 908 -374 1282 0.044	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979 1008 -312 1320 0.048	-90 870 864 860 10 3.0 0.098 0.977 1108 -250 1358 0.051 0.071 2.82	-90 870 864 859 11 3.4 0.098 0.974 1208 -187 1395 0.054 0.071 2.82	118 pc 100 pc 870 864 858 12 3.7 0.098 0.972 1308 -125 1433 0.057 0.071 2.82	-90 870 864 857 13 4.0 0.098 0.970 1408 -62 1470 0.059 0.071 2.82	870 864 856 14 4.3 0.098 0.967 1508 0 1508 0.062 0.071 2.82	870 ft 864 ft 855 ft 15 ft 4.6 m 0.098 g 0.965 1608 p: 62 p: 1546 p: 0.064 0.071 2.82
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  magnitude scaling factor (MSF) =	90 870 864 864 6 1.8 0.986 708 -499 1207 0.037 0.071	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984 808 -437 1245 0,041	-90 870 864 862 8 2.4 0.098 0.981 908 -374 1282 0.044	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979 1008 -312 1320 0.048	-90 870 864 860 10 3.0 0.098 0.977 1108 -250 1358 0.051	-90 870 864 859 11 3.4 0.098 0.974 1208 -187 1395 0.054	118 pc 100 pc 870 864 858 12 3.7 0.098 0.972 1308 -125 1433 0.057	-90 870 864 857 13 4.0 0.098 0.970 1408 -62 1470 0.059 0.071 2.82 1.00	870 864 856 14 4.3 0.098 0.967 1508 0 1508 0.062	850 ft 864 ft 855 ft 15 ft 4.6 ft 4.6 ft 4.6 ft 15 ft 62 pt 1546 pt 15
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  magnitude scaling factor (MSF) = ess correction factor (K_sigma) =	90 870 864 864 6 1.8 0.986 708 -499 1207 0.037 0.071 2.82	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984 808 -437 1245 0,041 0,071 2,82	-90 870 864 862 8 2.4 0.098 0.981 908 -374 1282 0.044 0.071 2.82	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979 1008 -312 1320 0.048 0.071 2.82	-90 870 864 860 10 3.0 0.098 0.977 1108 -250 1358 0.051 0.071 2.82	-90 870 864 859 11 3.4 0.098 0.974 1208 -187 1395 0.054 0.071 2.82	118 pc 100 pc 870 864 858 12 3.7 0.098 0.972 1308 -125 1433 0.057 0.071 2.82	-90 870 864 857 13 4.0 0.098 0.970 1408 -62 1470 0.059 0.071 2.82	870 864 856 14 4.3 0.098 0.967 1508 0.062 0.071 2.82 1.00 0.6	870 ft 864 ft 855 ft 4.6 m 0.098 g 0.965 1608 p 62 p 1546 p 0.064 0.071 2.82 1.00 0.6
Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =	gnitude = -90 870 864 864 6 1.8 0.986 708 -499 1207 0.037 0.071 2.82 1.00	5,00 M 0,098 g -90 870 864 863 7 2.1 0,098 0,984 808 -437 1245 0,041 0,071 2,82 1,00	-90 870 864 862 8 2.4 0.098 0.981 908 -374 1282 0.044 0.071 2.82 1.00	Unit weig Unit -90 870 864 861 9 2.7 0.098 0.979 1008 -312 1320 0.048 0.071 2.82 1.00	-90 870 864 860 10 3.0 0.098 0.977 1108 -250 1358 0.051 0.071 2.82 1.00	-90 870 864 859 11 3.4 0.098 0.974 1208 -187 1395 0.054	118 pc 100 pc 870 864 858 12 3.7 0.098 0.972 1308 -125 1433 0.057 0.071 2.82 1.00	-90 870 864 857 13 4.0 0.098 0.970 1408 -62 1470 0.059 0.071 2.82 1.00	870 864 856 14 4.3 0.098 0.967 1508 0.062 0.071 2.82 1.00	870 ft 864 ft 855 ft 15 ft 4.6 n 0.098 g 0.965 1608 p 62 p 1546 p 0.064 0.071 2.82 1.00

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Liquefaction Assessment.xls

Figure 16. Liquefaction Assessment in Vertical Profile 90 ft from Centerline of Dike at Sta. 527+80 for the Local Design Earthquake.

					_					_
Representative normalized	CPT tip res	sistance (q_	c1N)_cs =	12						
Dealer codhesial	2 200	AAPT KAI		Disease	ic surface e	Investion a	856 f			
Design earthqual		MEAN	i.							
Earthquake ma		6.32			ght of emba		118 p			
Peak ground acceleration	on rock =	0.053 g	,	UII	it weight of t	oai asn =	100 £	CI		
Offset from dike centerline =	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90 f
Elevation of ground surface ~	870	870	870	870	870	870	870	870	870	870 1
Elevation at top of ash ~	864	864	864	864	864	864	864	864	864	864 1
Elevation for assessment =	864	863	862	861	860	859	858	857	856	855 1
Depth below surface =	6	7	8	9	10	11	12	13	14	15 1
Depth below surface =	1.8	2.1	2.4	2.7	3.0	3.4	3.7	4.0	4.3	4.6
Assumed surface PGA =	0.053	0,053	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053
Stress reduction factor (r_d) =	0.986	0.984	0.981	0.979	0.977	0.974	0.972	0.970	0.967	0.965
Total vertical stress =	708	808	908	1008	1108	1208	1308	1408	1508	1608
Static pore water pressure =	-499	-437	-374	-312	-250	-187	-125	-62	0	62
Effective vertical stress =	1207	1245	1282	1320	1358	1395	1433	1470	1508	1546
Cyclic Stress Ratio (CSR) =	0.020	0.022	0.024	0,026	0.027	0.029	0.031	0.032	0.033	0.035
ance Ratio for 7.5 Mw (CRR_7.5) =	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
Magnitude scaling factor (MSF) =	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55
tress correction factor (K_sigma) =	1,00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
stress correction factor (K_alpha) =	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Cyclic Resistance Ratio (CRR) =	0.056	0.056	0.056	0.056	0.056	0.056	0.056	0.056	0.056	0.056
CONTRACTOR DESCRIPTION OF THE PROPERTY OF THE				1000	10.00		4 36 6	4 94	14.44	4.04
	2.80	2.54	2,33 1N) cs =	2.17	2.03	1,92	1.82	1.74	1.67	1.61
Representative normalized C Design earthquake Earthquake mag	CPT tip resi	stance (q_c MEAN 6.32 M	1N)_cs =	25 Phreatl Unit welg	c surface ele	evation = kment =	856 ft 118 pc	ef .	1,67	1.01
Representative normalized C	CPT tip resi	stance (q_c	1N)_cs =	25 Phreatl Unit welg	c surface ele	evation = kment =	856 ft	ef .	1,67	1,01
Representative normalized C Design earthquake Earthquake mag	CPT tip resi	stance (q_c MEAN 6.32 M	1N)_cs =	25 Phreatl Unit welg	c surface ele	evation = kment =	856 ft 118 pc	ef .	-90	-90 ft
Representative normalized C Design earthquake Earthquake mag Peak ground acceleration o	CPT tip resi e event = gnitude = on rock =	MEAN 6.32 M 0.053 g	1N)_cs = <mark>.</mark> w	25 Phreati Unit welg Unit	c surface eld ht of embar weight of co	evation =    kment =  aal ash =	<b>856</b> ft 118 pc 100 pc	a a	740	-90 ft
Representative normalized of Design earthquake Earthquake mag Peak ground acceleration of Offset from dike centerline =	CPT tip resise event = gnitude = on rock =	MEAN 6.32 M 0.053 g	1N)_cs = w -90	25 Phreati Unit welg Unit	c surface eld ht of embar weight of co	evation =    kment =  pal ash =	856 ft 118 pc 100 pc	:f :f -90	-90	-90 ft 870 ft
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~	CPT tip resise event = gnitude = on rock = -90 870	MEAN 6.32 M 0.053 g	1N)_cs = w -90 870	25 Phreath Unit welg Unit -90 870	c surface eld int of embar weight of co -90 870	evation =   	856 ft 118 pc 100 pc -90 870	ef ef -90 870	-90 870	-90 ft 870 ft 864 ft 855 ft
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~	e event = gnitude = on rock = -90 870 864	MEAN 6.32 M 0.053 g -90 870 864	1N)_cs = w -90 870 864	25 Phreath Unit weig Unit -90 870 864	c surface ele ht of embar weight of co -90 870 864	evation =   	856 ft 118 pc 100 pc -90 870 864	-90 870 864	-90 870 864	-90 ft 870 ft 864 ft 855 ft
Representative normalized of Design earthquake Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment =	e event = gnitude = gn rock = -90 870 864	MEAN 6.32 M 0.053 g -90 870 864	1N)_cs =w  -90 870 864 862	25 Phreath Unit welg Unit -90 870 864	c surface ele ht of embar weight of co -90 870 864 860	evation =   	856 ft 118 pc 100 pc -90 870 864 858	-90 870 864 857	-90 870 864 856	-90 fi 870 fi 864 fi 855 fi 15 fi
Representative normalized of Design earthquake Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface =	e event = gnitude = pn rock = -90 870 864 864 6	MEAN 6.32 M 0.053 g -90 870 864 863 7	1N)_cs =w  -90 870 864 862 8	25 Phreath Unit welg Unit -90 870 864 861	c surface eld int of embar weight of co -90 870 864 860 10	evation =	856 ft 118 pc 100 pc -90 870 864 858 12	-90 870 864 857 13 4.0	-90 870 864 856	-90 ft 870 ft 864 ft 855 ft 15 ft 4.6 m
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface =	e event = gnitude = pn rock = -90 870 864 6 1.8	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1	-90 870 864 862 8	25  Phreath Unit welg Unit -90 870 864 861 9 2.7	c surface eld ht of embar weight of co -90 870 864 860 10 3.0	evation =	856 ft 118 pc 100 pc -90 870 864 858 12 3.7	-90 870 864 857 13 4.0	-90 870 864 856 14 4.3	-90 ft 870 ft 864 ft 15 ft 4.6 m
Representative normalized of Design earthquake Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA =	e event = gnitude = gnitude = -90 870 864 864 6 1.8 0.053	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1	-90 870 864 862 8 2.4	25 Phreatic Unit weld Unit weld Unit -90 870 864 861 9 2.7 0.053	c surface ela ht of embar weight of co -90 870 864 860 10 3.0 0.053 0.977 1108	evation =	856 ft 118 pc 100 pc -90 870 864 858 12 3.7 0.053 0.972 1308	-90 870 864 857 13 4.0 0.053 0,970	-90 870 864 856 14 4.3 0.053 0.967 1508	-90 ft 870 ft 864 ft 855 ft 4.6 m 0.053 g 0.965 1608 pt
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) =	e event = gnitude = son rock = -90 870 864 864 6 1.8 0.053 0.986	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808 -437	-90 870 864 862 8 2.4 0.053 0.981	25 Phreath Unit welg Unit -90 870 864 861 9 2.7 0.053 0.979	c surface eld the of embar weight of co -90 870 864 860 10 3.0 0.053 0.977 1108 -250	evation =	856 ft 118 pc 100 pc 870 864 858 12 3.7 0.053 0.972 1308 -125	-90 870 864 857 13 4.0 0.053 0.970 1408 -62	-90 870 864 856 14 4.3 0.053 0.967 1508	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 p; 62 p;
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress =	e event = gnitude = gnitude = gnitude = -90 870 864 864 6 1.8 0.053 0.986 708	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808	-90 870 864 862 8 2.4 0.053 0.981 908	25  Phreath Unit welg Unit  -90 870 864 861 9 2.7 0.053 0.979 1008	-90 870 864 860 10 3.0 0.053 0.977 1108 -250 1358	evation =	856 ft 118 pc 100 pc -90 870 864 858 12 3.7 0.053 0.972 1308	-90 870 864 857 13 4.0 0.053 0,970 1408 -62 1470	-90 870 864 856 14 4.3 0.053 0.967 1508	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 pp 1546 pp
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure =	e event = gnitude = son rock = -90 870 864 6 1.8 0.053 0.986 708 -499	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808 -437	-90 B70 B64 B62 B 2.4 0.053 0.981 908 -374	25  Phreath Unit welg Unit  -90 870 864  861 9 2.7  0.053 0.979 1008 -312	c surface eld the of embar weight of co -90 870 864 860 10 3.0 0.053 0.977 1108 -250	evation =	856 ft 118 pc 100 pc 870 864 858 12 3.7 0.053 0.972 1308 -125	-90 870 864 857 13 4.0 0.053 0.970 1408 -62	-90 870 864 856 14 4.3 0.053 0.967 1508	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 p; 62 p;
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  nce Ratio for 7.5 Mw (CRR_7.5) =	e event = gnitude = gnitud	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808 -437 1245 0.022	-90 B70 864 862 8 2.4 0.053 0.981 908 -374 1282 0.024	25  Phreath Unit welg Unit  -90 870 864  861 9 2.7  0.053 0.979 1008 -312 1320 0.026  0.071	surface ela ht of embar weight of co -90 870 864 860 10 3.0 0.053 0.977 1108 -250 1358 0.027	evation =	856 ft 118 pc 100 pc 870 864 858 12 3.7 0.053 0.972 1308 -125 1433 0.031	-90 870 864 857 13 4.0 0.053 0.970 1408 -62 1470 0.032	-90 870 864 856 14 4.3 0.053 0.967 1508 0 1508 0.033	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 pt 62 pt 1546 pt 0.035
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  nce Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) =	e event = gnitude = gnitud	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808 -437 1245 0.022	-90 870 864 862 8 2.4 0.053 0.981 908 -374 1282 0.024 0.071 1.55	25  Phreath Unit welg Unit  -90 870 864  861 9 2.7  0.053 0.979 1008 -312 1320 0.026  0.071 1.55	-90 870 864 860 10 3.0 0.053 0.977 1108 -250 1358 0.027	evation =	856 ft 118 pc 100 pc 870 864 858 12 3.7 0.053 0.972 1308 -125 1433 0.031	-90 870 864 857 13 4.0 0.053 0,970 1408 -62 1470 0.032	-90 870 864 856 14 4.3 0.053 0.967 1508 0.033 0.071 1.55	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 pr 62 pr 1546 pr 0.035
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  nce Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) = ress correction factor (K_sigma) =	e event = gnitude = son rock = -90 870 864 6 1.8 0.053 0.986 708 -499 1207 0.020 0.071 1.55 1.00	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808 -437 1245 0.022	-90 870 864 862 8 2.4 0.053 0.981 908 -374 1282 0.024 0.071 1.55 1.00	25  Phreatic Unit weigners with the second s	c surface ele ht of embar weight of co  -90 870 864 860 10 3.0 0.053 0.977 1108 -250 1358 0.027 0.071 1.55 1.00	evation =	856 ft 118 pc 100 pc 870 864 858 12 3.7 0.053 0.972 1308 -125 1433 0.031	-90 870 864 857 13 4.0 0.053 0.970 1408 -62 1470 0.032	-90 870 864 856 14 4.3 0.053 0.967 1508 0.033 0.071 1.55 1.00	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 pt 62 pt 1546 pt 0.035 0.071 1.55 1.00
Representative normalized C  Design earthquake Earthquake mag Peak ground acceleration of  Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =  more Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) = ress correction factor (K_sigma) = tress correction factor (K_gigma) =	e event = gnitude = son rock = -90 870 864 864 6 1.8 0.053 0.986 708 -499 1207 0.020 0.071 1.55 1.00 0.6	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808 -437 1245 0.022 0.071 1.55 1.00 0.6	-90 870 864 862 8 2.4 0.053 0.981 908 -374 1282 0.024 0.071 1.55 1.00 0.6	25  Phreatil Unit weight of the property of th	c surface ele ht of embar weight of co  -90 870 864  860 10 3.0  0.053 0.977 1108 -250 1358 0.027  0.071 1.55 1.00 0.6	evation =	856 ft 118 pc 100 pc 870 864 858 12 3.7 0.053 0.972 1308 -125 1433 0.031 0.071 1.55 1.00 0.6	-90 870 864 857 13 4.0 0.053 0.970 1408 -62 1470 0.032 0.071 1.55 1.00 0.6	-90 870 864 856 14 4.3 0.053 0.967 1508 0.033 0.071 1.55 1.00 0.6	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 pt 1546 pt 0.035 0.071 1.55 1.00 0.6
Design earthquake Earthquake mag Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface = Assumed surface r_d) = Total vertical stress = Stress reduction factor (r_d) = Total vertical stress = Effective vertical stress = Cyclic Stress Ratio (CSR) =  Ince Ratio for 7.5 Mw (CRR_7.5) =	e event = gnitude = son rock = -90 870 864 6 1.8 0.053 0.986 708 -499 1207 0.020 0.071 1.55 1.00	MEAN 6.32 M 0.053 g -90 870 864 863 7 2.1 0.053 0.984 808 -437 1245 0.022	-90 870 864 862 8 2.4 0.053 0.981 908 -374 1282 0.024 0.071 1.55 1.00	25  Phreatic Unit weigners with the second s	c surface ele ht of embar weight of co  -90 870 864 860 10 3.0 0.053 0.977 1108 -250 1358 0.027 0.071 1.55 1.00	evation =	856 ft 118 pc 100 pc 870 864 858 12 3.7 0.053 0.972 1308 -125 1433 0.031	-90 870 864 857 13 4.0 0.053 0.970 1408 -62 1470 0.032	-90 870 864 856 14 4.3 0.053 0.967 1508 0.033 0.071 1.55 1.00	-90 ft 870 ft 864 ft 15 ft 4.6 m 0.053 g 0.965 1608 pt 62 pt 1546 pt 0.035 0.071 1.55 1.00

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Liquefaction Assessment.xls

Figure 17. Liquefaction Assessment in Vertical Profile 90 ft from Centerline of Dike at Sta. 527+80 for the Mean Design Earthquake.

Design earthquak	e event -	MODAL		Phreni	ic surface e	levation =	856 ff			
Earthquake ma		7.70 N	Au		ght of emba		118 p			
Peak ground acceleration		0.037			t weight of c		100 p			
1318 1313 11111 1111	2002207				0.00		0.000			
Offset from dike centerline =	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90 fi
Elevation of ground surface ~	870	870	870	870	870	870	870	870	870	870 fi
Elevation at top of ash ~	864	864	864	864	864	864	864	864	864	864 ft
Elevation for assessment =	864	863	862	861	860	859	858	857	856	855 fi
Depth below surface =	6	7	8	9	10	11	12	13	14	15 f
Depth below surface =	1.8	2.1	2.4	2.7	3.0	3,4	3.7	4.0	4.3	4.6 n
Assumed surface PGA =	0.037	0.037	0.037	0.037	0.037	0.037	0.037	0.037	0,037	0.037 g
Stress reduction factor (r_d) =	0.986	0.984	0.981	0.979	0.977	0.974	0.972	0.970	0.967	0.965
Total vertical stress =	708	808	908	100B	1108	1208	1308	1408	1508	1608 p
Static pore water pressure =	-499	-437	-374	-312	-250	-187	-125	-62	0	62 p
Effective vertical stress =	1207	1245	1282	1320	1358	1395	1433	1470	1508	1546 p
Cyclic Stress Ratio (CSR) =	0.014	0.015	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024
ince Ratio for 7.5 Mw (CRR_7.5) =	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
Magnitude scaling factor (MSF) =	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
tress correction factor (K_sigma) =	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
tress correction factor (K_alpha) =	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Cyclic Resistance Ratio (CRR) =	0.034	0.034	0.034	0.034	0.034	0.034	0.034	0.034	0.034	0.034
fety against liquefaction (FS_liq) =	2.42	2.19	2.01	1.87	1.75	1.66	1.58	1.51	1.45	1.39
Design earthquake	e event =	MODAL		Discoul.						
Earthquake mag Peak ground acceleration o		7.70 M 0.037 g	w	Unit weig	c surface ele ht of embar weight of co	kment =	856 ft 118 pc 100 pc			
			w	Unit weig	ht of embar	kment =	118 pc			
			-90	Unit weig	ht of embar	nkment = oal ash = -90	118 pc	ef -90	-90	-90 ft
Peak ground acceleration of	on rock =	0.037 g		Unit weig Unit	ht of embar weight of co	nkment = oal ash ≃	118 pc 100 pc	-90 870	870	870 ft
Peak ground acceleration of Offset from dike centerline	on rock = -90	0.037 g -90	-90	Unit weig Unit -90	ht of embar weight of co	nkment = oal ash = -90	118 pc 100 pc	ef -90		870 ft
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~	-90 870	0.037 g -90 870	-90 870	Unit weig Unit -90 870	ht of embar weight of co -90 870	nkment = oal ash = -90 870	118 pc 100 pc -90 870	-90 870	870	870 ft 864 ft 855 ft
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~	-90 870 864	-90 870 864	-90 870 864	Unit weig Unit -90 870 864	ht of embar weight of co -90 870 864	-90 870 864 859	-90 870 864 858	-90 870 864	870 864 856 14	870 ft 864 ft 855 ft 15 ft
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment =	-90 870 864	-90 870 864 863	-90 870 864 862	Unit weig Unit -90 870 864 861	ht of embar weight of co -90 870 864 860	nkment = cal ash = -90 870 864 859	118 pc 100 pc -90 870 864	-90 870 864 857	870 864 856	870 ft 864 ft 855 ft 15 ft
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface =	-90 870 864 864	0.037 g -90 870 864 863 7	-90 870 864 862 8	Unit weig Unit -90 870 864 861 9	-90 870 864 880 10 3.0	-90 870 864 859 11 3.4	118 pc 100 pc -90 870 864 858 12 3.7	-90 870 864 857 13 4.0	870 864 856 14 4.3	870 ft 864 ft 855 ft 15 ft 4.6 m
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface =	-90 870 864 864 6 1.8	-90 870 864 863 7 2.1	-90 870 864 862 8	Unit weig Unit -90 870 864 861 9 2.7	-90 870 864 860 10 3.0 0.037 0.977	-90 870 864 859 11 3.4	-90 870 864 858 12 3.7	-90 870 864 857 13 4.0 0.037 0.970	856 14 4.3 0.037 0.967	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA =	90 870 864 864 6 1.8	0.037 g -90 870 864 863 7 2.1	-90 870 864 862 8 2.4	Unit weig Unit -90 870 864 861 9 2.7	-90 870 864 880 10 3.0	-90 870 864 859 11 3.4	118 pc 100 pc -90 870 864 858 12 3.7	-90 870 864 857 13 4.0	870 864 856 14 4.3	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965 1608 ps
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~  Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) =	90 870 864 864 6 1.8 0.037 0.986	0.037 g -90 870 864 863 7 2.1 0.037 0.984	-90 870 864 862 8 2.4 0.037 0.981	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979	-90 870 864 860 10 3.0 0.037 0.977 1108 -250	-90 870 864 859 11 3.4 0.037 0.974	118 pc 100 pc 870 864 858 12 3.7 0.037 0.972	-90 870 864 857 13 4.0 0.037 0.970 1408 -62	870 864 856 14 4.3 0.037 0.967 1508	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965 1608 ps 62 ps
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress =	90 870 864 864 6 1.8 0.037 0.986 708	0.037 g -90 870 864 863 7 2.1 0.037 0.984 808 -437 1245	-90 870 864 862 8 2.4 0.037 0.981 908	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979 1008	-90 870 864 860 10 3.0 0.037 0.977 1108 -250 1358	-90 870 864 859 11 3.4 0.037 0.974 1208 -187 1395	118 pc 100 pc 870 864 858 12 3.7 0.037 0.972 1308 -125 1433	-90 870 864 857 13 4.0 0.037 0.970 1408 -62 1470	870 864 856 14 4.3 0.037 0.967 1508 0	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965 1608 ps 62 ps 1546 ps
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure =	-90 870 864 864 6 1.8 0.037 0.986 708 -499	0.037 g -90 870 864 863 7 2.1 0.037 0.984 808 -437	-90 870 864 862 8 2.4 0.037 0.981 908 -374	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979 1008 -312	-90 870 864 860 10 3.0 0.037 0.977 1108 -250	-90 870 864 859 11 3.4 0.037 0.974 1208 -187	118 pc 100 pc 870 864 858 12 3.7 0.037 0.972 1308 -125	-90 870 864 857 13 4.0 0.037 0.970 1408 -62	870 864 856 14 4.3 0.037 0.967 1508	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965 1608 ps 62 ps
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Estress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) =	-90 870 864 864 6 1.8 0.037 0.986 708 -499 1207	0.037 g -90 870 864 863 7 2.1 0.037 0.984 808 -437 1245	-90 870 864 862 8 2.4 0.037 0.981 908 -374 1282	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979 1008 -312 1320	-90 870 864 860 10 3.0 0.037 0.977 1108 -250 1358	-90 870 864 859 11 3.4 0.037 0.974 1208 -187 1395	118 pc 100 pc 870 864 858 12 3.7 0.037 0.972 1308 -125 1433	-90 870 864 857 13 4.0 0.037 0.970 1408 -62 1470 0.022	870 864 856 14 4.3 0.037 0.967 1508 0	870 ft 864 ft 15 ft 4.6 m 0.037 g 0.965 1608 p: 1546 p
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) = nce Ratio for 7.5 Mw (CRR_7.5) =	-90 870 864 864 6 1.8 0.037 0.986 -499 1207 0.014	0.037 g -90 870 864 863 7 2.1 0.037 0.984 808 -437 1245 0.016	-90 870 864 862 8 2.4 0.037 0.981 908 -374 1282 0.017	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979 1008 -312 1320 0.018	-90 870 864 860 10 3.0 0.037 0.977 1108 -250 1358 0.019 0.071	-90 870 864 859 11 3.4 0.037 0.974 1208 -187 1395 0.020	-90 870 864 858 12 3.7 0.037 0.972 1308 -125 1433 0.021	-90 870 864 857 13 4.0 0.037 0.970 1408 -62 1470 0.022 0.071 0.93	870 864 856 14 4.3 0.037 0.967 1508 0 1508 0.023 0.071 0.93	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965 1608 ps 62 ps 1546 ps 0.024 0.071 0.93
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Effective vertical stress = Effective vertical stress =	-90 870 864 864 6 1.8 0.037 0.986 708 -499 1207 0.014	0.037 g -90 870 864 863 7 2.1 0.037 0.984 808 -437 1245 0.015	-90 870 864 862 8 2.4 0.037 0.981 908 -374 1282 0.017	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979 1008 -312 1320 0.018	-90 870 864 860 10 3.0 0.037 0.977 1108 -250 1358 0.019	-90 870 864 859 11 3.4 0.037 0.974 1208 -187 1395 0.020	118 pc 100 pc 870 864 858 12 3.7 0.037 0.972 1308 -125 1433 0.021	-90 870 864 857 13 4.0 0.037 0.970 1408 -62 1470 0.022 0.071 0.93 1.00	870 864 856 14 4.3 0.037 0.967 1508 0.023 0.071 0.93 1.00	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965 1608 pt 52 pt 1546 pt 0.024 0.071 0.93 1.00
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Static pore water pressure = Effective vertical stress = Cyclic Stress Ratio (CSR) = nce Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) =	-90 870 864 864 6 1.8 0.037 0.986 708 -499 1207 0.014 0.071 0.93	0.037 g -90 870 864 863 7 2.1 0.037 0.984 808 -437 1245 0.015 0.071 0.93	-90 870 864 862 8 2.4 0.037 0.981 908 -374 1282 0.017 0.071 0.93	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979 1008 -312 1320 0.018 0.071	-90 870 864 860 10 3.0 0.037 0.977 1108 -250 1358 0.019 0.071	-90 870 864 859 11 3.4 0.037 0.974 1208 -187 1395 0.020	118 pc 100 pc 870 864 858 12 3.7 0.037 0.972 1308 -125 1433 0.021 0.071 0.93 1.00 0.6	-90 870 864 857 13 4.0 0.037 0.970 1408 -62 1470 0.022 0.071 0.93 1.00 0.6	870 864 856 14 4.3 0.037 0.967 1508 0 1508 0.023 0.071 0.93 1.00 0.6	870 ft 864 ft 15 ft 4.6 m 0.037 g 0.965 1608 pt 62 pt 1546 pt 0.024 0.071 0.93 1.00 0.8
Peak ground acceleration of Offset from dike centerline = Elevation of ground surface ~ Elevation at top of ash ~ Elevation for assessment = Depth below surface = Depth below surface = Depth below surface = Assumed surface PGA = Stress reduction factor (r_d) = Total vertical stress = Effective vertical stress = Cyclic Stress Ratio (CSR) = more Ratio for 7.5 Mw (CRR_7.5) = Magnitude scaling factor (MSF) = ess correction factor (K_sigma) =	-90 870 864 864 6 1.8 0.037 0.986 708 -499 1207 0.071 0.93 1.00	0.037 g -90 870 864 863 7 2.1 0.037 0.984 808 -437 1245 0.015 0.071 0.93 1.00	-90 870 864 862 8 2.4 0.037 0.981 908 -374 1282 0.017 0.071 0.93 1.00	Unit weig Unit -90 870 864 861 9 2.7 0.037 0.979 1008 -312 1320 0.018	-90 870 864 860 10 3.0 0.037 0.977 1108 -250 1358 0.019 0.071 0.93 1.00	-90 870 864 859 11 3.4 0.037 0.974 1208 -187 1395 0.020	118 pc 100 pc 870 864 858 12 3.7 0.037 0.972 1308 -125 1433 0.021	-90 870 864 857 13 4.0 0.037 0.970 1408 -62 1470 0.022 0.071 0.93 1.00	870 864 856 14 4.3 0.037 0.967 1508 0.023 0.071 0.93 1.00	870 ft 864 ft 855 ft 15 ft 4.6 m 0.037 g 0.965 1608 pr 1546 pr 1546 pr 1546 pr 1540 pr

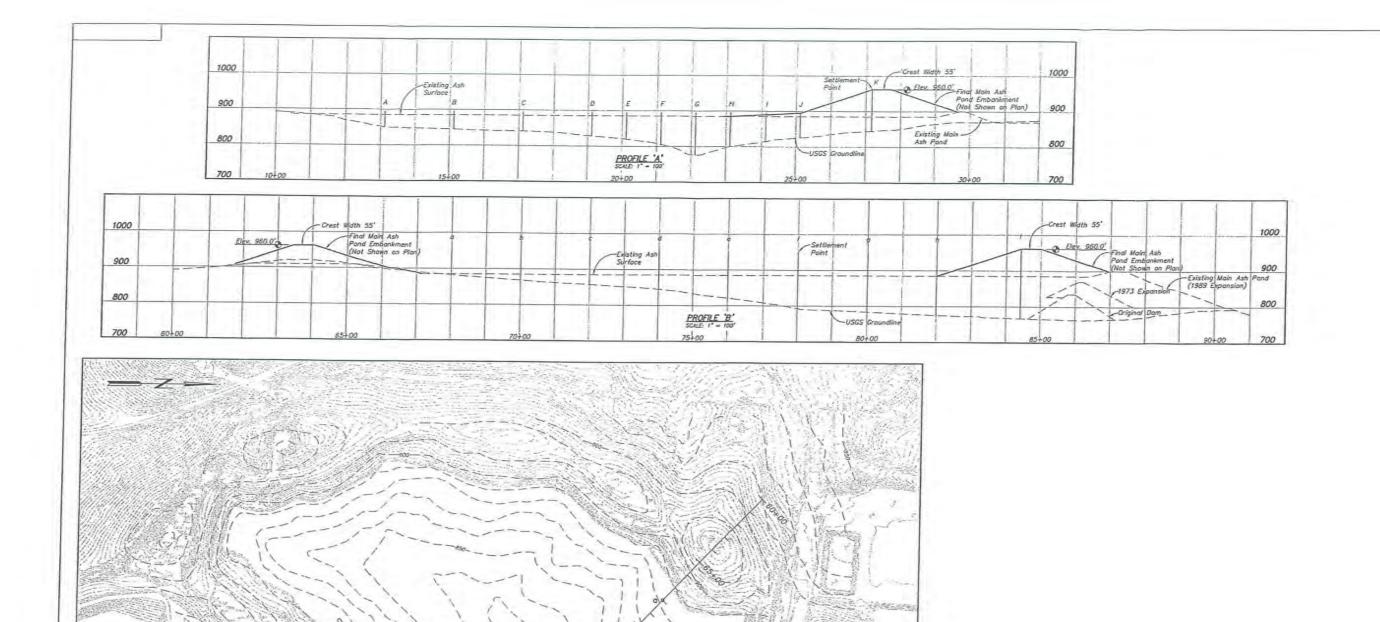
Fuller Mossbarger Scott &



Liquefaction Assessment.xls

Figure 18. Liquefaction Assessment in Vertical Profile 90 ft from Centerline of Dike at Sta. 527+80 for the Modal Design Earthquake.

Appendix F
Settlement Calculations



Existing Main Ash Pond

LEGEND

Analysis Location

Topographic information is based an aerial mapping performed by L. Kimball and Associates and others. Refer to General Notes and Notes on Sneet 803 through 812 for Main Ash Pond-Starter Dike.

		REVISIONS			S				THE				
E.W. BROWN GENERATING STATION    Design Cold   Design Cold		h 3-1		See Steen Steen Steen Sty Statistics into:				SETTLEMENT PLAN & PROFILE					S
Drawn, CSI													
	Rankover F S					Drawn Sele 875 Checket	004 000 000			Utili Com	ties		
2007-1000 -0000 -0 00-15 00-150000 (195000)		H				JOS MO. 119963	JOB NO	108 NO	DN 804.0	Dresty No.	3m		

PLAN - SETTLEMENT ANALYSIS

scv LX2006193 11/27/2006

f Parameters	71 Height	109 67	110
	OB 30		0.28 1.1
Summary of Par	Description	Gypsum	Flv Ash
	No.	1	2

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma'_{vc}}{\sigma'_{vo}}$$

Sublayer	(inches)	14.3	9,5	7.3	6,1	6.3	4.7	4.2
Sublayer	(inches)	09	09	09	99	9	9	90
Change in Void Ratio (Δe)		0.5100	0.3405	0.2596	0.2161	0.1873	0.1662	0.1499
Final Vertical Effective Stress	(psd)	7890	8440	8990	9540	10090	10640	11190
Final Pore Water Pressure	(bst)	0	0	0	0	0	0	0
Final Total Vertical Stress	(pst)	275	825	1375	1925	2475	3025	3575
Added Pressure due to Embankment Loading	(bsd)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Added Pressure due to 5' water		312	312	312	312	312	312	312
Added FIII Pressure	(bsd)	7303	7303	7303	7303	7303	7303	7303
Initial Vertical Effective Stress	(Jsd)	119	513	1063	1613	2163	27.13	3263
Initial Pore Water Pressure		156	312	312	312	312	312	312
Initial Total Vertical Stress	(Jsd)	275	825	1375	1925	2475	3025	3575
Total Unit Weight	(bct)	110	110	110	110	110	110	110
Void Ratio		1.14	1.14	1.14	1.14	1.14	1,14	1.14
Layer Midpoint Elevation	(H)	882.5	877.5	872.5	867.5	862.5	857.5	852.5
Layer Top Elevation	(#)	880	875	870	965	860	855	850
Layer No.		÷	2	69	4	w	iD.	7

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

11/27/2006 SCV

scv LX2006193

 No.
 Description
 Cc
 eo
 Yt
 Height

 1
 Gypsum
 0.28
 1.14
 110
 67

Expression:

 $\Delta H = H \left( \frac{C_{\epsilon}}{1 + \epsilon_{\sigma}} \right) \log \frac{\sigma' \frac{w}{w}}{\sigma' v_{\sigma}}$ 

Sublayer Compression (inches)	14.3	8.5	7,3	5.1	5,3	4.7	4.2	3.8
Sublayer Thickness (inches)	09	90	90	09	99	09	9	09
Change in Vold Ratio (Δe)	0.5100	0.3405	0.2596	0,2161	0.1873	0.1662	0.1499	0.1368
Final Vertical Effective Stress (psf)	7890	8440	8990	9540	10090	10640	11190	11740
Final Pore Water Pressure (psf)	0	0	0	D	0	b	D	0
Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125
Added Pressure due to Embankment Loading (psf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Added Pressure due to 5' water (psf)	312	312	312	312	312	312	312	312
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	7303	7303
Initial Vertical Effective Stress (psf)	119	513	1063	1613	2163	2713	3263	3813
Initial Pore Water Pressure (psf)	156	312	312	312	312	312	312	312
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125
Total Unit Weight (pcf)								110
Void Ratio	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867.5	862.5	857,5	852,5	847.5
Layer Top Elevation (ft) 885	880	875	870	865	960	855	850	845
Layer No.		2	es	4	w	ω	1	60

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

11/27/2006 SCV LX2006193

		Helph	100	5
		Ye	100	0++
	Parameters	08		1.44
		nmary of Parameters	S	
Sections	Suffittingly of Pe	Description	Gypsum	Fiv Ash
		No.	1	2

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma'_{v_o}}{\sigma'_{v_o}}$$

	Sublayer Compression (inches)	14.3	9,5	7.3	1,1	6.3	4.7	4.2	3.8	3,5	
	Sublayer Thickness (inches)	09	90	9	90	09	90	90	90	09	
	Change in Void Ratio (Δe)	0.5100	0.3405	0.2596	0.2161	0.1873	0.1662	0.1499	0.1368	0,1259	
	Final Vertical Effective Stress (psf)	7890	8440	8990	9540	10090	10640	11190	11740	12290	
	Final Pore Water Pressure (psf)	0	0	0	0	0	0	0	0	0	
	Final Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4575	
Added	Pressure due to Embankment. Loading (psf)	0.00	0.00	0.00	0.00	00'0	0.00	0.00	00.00	0.00	
	Added Pressure due to 5' water (psf)	312	312	312	312	312	312	312	312	312	
	Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	7303	7303	7303	
	Initial Vertical Effective Stress (psf)	119	513	1063	1613	2163	2713	3263	3813	4363	
	Initial Pore Water Pressure (psf)	156	312	312	312	312	312	312	312	312	
	Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	
	Total Unit Weight (pcf)	110	110	110	110	110	110	110	110	110	
	Void Ratio										
	Layer Midpoint Elevation (ft)										
	Layer Top Elevation (ft) 885	880	875	870	865	860	855	850	845	840	
	Layer No.	-	2	m	4	w	ω	7	ю	on	

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

11/27/2006

scv LX2006193

0.28 1.14 60 Summary of Parameters
Description Cc
Gypsum
Fly Ash log G'y  $\frac{C_{\epsilon}}{1+e_{o}}$  $\Delta H = H$ Expression:

No.

Height 67

Sublayer Compression (inches)	14.3	9.5	7.3	6.1	5.3	4.7	4.2	3.8	3.5	3.3	3.1
Sublayer Thickness (inches)	09	9	9	99	09	09	90	09	90	90	60
Change in Void Ratio (Δe)	0,5100	0.3405	0.2596	0.2161	0.1873	0.1662	0,1499	0.1368	0.1259	0.1168	0.1090
Final Vertical Effective Stress (psf)	7890	8440	9980	9540	10090	10640	11190	11740	12290	12840	13390
Final Pore Water Pressure (psf)	0	0	0	0	0	0	0	0	0	0	0
Final Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775
Added Pressure due to Embankment Loading (psf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Added Pressure due to 5' water (psf)	312	312	312	312	312	312	312	312	312	312	312
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303
Initial Vertical Effective Stress (psf)	119	513	1063	1613	2163	2713	3263	3813	4363	4913	5463
Initial Pore Water Pressure (psf)	156	312	312	312	312	312	312	312	312	312	312
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775
Total Unit Weight (pcf)	110	110	110	110	110	110	110	110	110	110	110
Void Ratio	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14
Layer Midpoint Elevation (ft)	882,5	877.5	872.5	867,5	862.5	857.5	852.5	847,5	842.5	837.5	832.5
Layer Top Elevation (ft) 885	880	875	870	865	980	855	850	845	840	835	830
Layer No.	+	24	m	4	uş.	ш	7	œ	on	10	Ŧ

65.0

Total Settlement

scv LX2006193

	Helght	19	
	¥	109	440
	08		444
arameters	သ		86.0
Summary of Para	Description	Gypsum	Flv Ach
	No.	1	2

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma' _{\rm w}}{\sigma' _{\rm wo}}$$

														inches
Sublayer Compression (inches)	14.3	6	7.3	6.1	5.3	1.7	4.2	3.8	3,5	65	3.1	2.9	2.7	70.6
Sublayer Thickness (inches)	09	09	09	00	09	09	90	09	09	09	90	09	90	
Change in Void Ratio (∆e)	0.5100	0.3405	0.2596	0.2161	0.1873	0.1662	0.1499	0.1368	0.1259	0.1168	0,1090	0.1022	0.0963	Total Settlement
Final Vertical Effective Stress (psf)	7890	8440	8990	9540	10090	10640	11190	11740	12290	12840	13390	13940	14490	Total S
Final Pore Water Pressure (psf)	0	o	0	0	0	0	0	0	0	0	0	0	0	
Final Total Final Pore Vertical Water Stress Pressure (psf) (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875	
Added Pressure due to Embankment Loading (psf)	0.00	0.00	0.00	0,00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Added Pressure due to 5' water (psf)	312	312	312	312	312	312	312	312	312	312	312	312	312	
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	
Initial Vertical Effective Stress (psf)	119	53	1063	1613	2163	2713	3263	3813	4363	4913	5463	6013	6563	
Initial Pore Water Pressure (psf)	136	312	312	312	312	312	312	312	312	312	312	312	312	
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875	
Total Unit Weight (pcl)	110	110	110	110	110	110	110	110	110	110	110	110	110	
Void Ratio	1.14	134	1.14	1:14	1,14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867,5	862.5	857.5	852.5	847.5	842.5	837.5	832.5	827.5	822.5	
Layer Top Elevation (ft)	880	875	870	865	860	855	850	845	840	835	830	825	820	
Layer No.	-	61	ts.	4	ın	ω	7	60	m	9	#	12	6.3	

inches

scv LX2006193

Sublayer Sublayer
Thickness Compression
(inches) (inches)

14.3 9.5 7.3 6.1 4.7

09 09 09

2.9

90

9

2.6

9

2.7

8 8

80 80 80 E0 80 80 80 E0

9 9

90

9 8

77.8

Total Settlement

2.3

2.4

9 9

E.W. Brown Ash Pond Modification Kentucky Utilities -Results of Settlement Analysis

11/27/2006 scv LX2006193

Height 67 Summary of Parameters
Description Cc 60

0.28 1.14

Gypsum Fly Ash

 $\Delta H = H \left( \frac{C_c}{1 + e_c} \right) \log \frac{\sigma^*_{s,s}}{\sigma^*_{s,s}}$ 

Expression:

Final
Vertical Change in
Effective Void Ratio
Stress (Δe) Final Total Final Pore Vertical Water E Stress Pressure Added
Pressure due
to
Embankment
Loading Added Pressure due to 5' (bsd)

Added FIII

Initial Vertical Effective Stress (psf)

Initial Pore Water Pressure (psf)

Total Vertical Stress (psf)

Layer Midpoint Elevation

Layer Top Elevation

E

(4)

(psd)

14.3 275

Sublayer Thickness (inches)

0.5100

0.3405

8440

825 1375 1925

00'0 0.00 0.00 0.00 000 000

312

7303 7303

156 312 312

312 312 312

513

825 1375

110

877.5

8725 857.5

8 0.2596 0.2161

13

9.5

6.1 5.3 4.7

10090

312

2163

312

2475

110

9540

0.1873 0,1862

11190 11740

000

312

4125

337.5

832.5

842.5

852.5 847.5 0.00

312

0.1259 0.1358

0.1168

12290

0.1090

12840 13390 13940

0.1022

20

0,0911 0.0854

15040

7425

00.0 0.00

7425

822.5 817.5 812.5

99 9

> 0.0783 0.0749 0.0717 0.0588

0.00 0.00 000

312

10175 10725 11275

110

792,5

787,5 782,5

9825

000

8213

110

807.5

802.5 797.5

17240 17790 18340

0.4 8 20 0.0662

18890 19440 Total S

11275 11825

000 000

7303 7303

11513

312

11325

110

777.5

780

8

09

3.8 0.0637

ø

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

11/27/2006 scv LX2006193

No.         Description         Cc         eo         Y           1         Gypsum         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         11         12	No. Description Cc ea
Gypsum Fly Ash	1 Gypsum
Fly Ash	
Ynrassion	2 Fly Ash 0.28 1,14
	Expression:

Height 67

																		inches
(inches)	14.3	8.9	7.3	6.1	5.3	4.7	4.2	65 65	3.5	3.3	3.1	2.9	2.7	2.6	2.4	2.3	2.2	0.08
(inches)	09	90	90	90	90	90	90	9	09	09	90	09	09	90	09	9	9	
	0.5100	0,3405	0,2596	0.2161	0.1873	0.1662	0.1499	0.1368	0.1259	0.1168	0.1090	0.1022	0.0963	0.0911	0.0864	0.0822	0.0783	Total Settlement
(bst)	7890	8440	8990	9540	10090	10640	11190	11740	12290	12840	13390	13940	14490	15040	15590	16140	16690	Total Se
(bst)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
(bst)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875	7425	7975	8525	9075	
(pst)	0.00	0.00	0.00	00'0	00'0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
(pst)	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	
(bst)	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	
(bst)	119	513	1063	1613	2163	2713	3263	3813	4363	4913	5463	6013	8563	7113	7663	8213	8763	
(bsd)	156	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	
(isd)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875	7425	7975	8525	9075	
(ma)	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	
ful	882.5	877.5	872.5	867.5	862.5	857.5	852.5	847.5	842.5	837.5	832.5	827.5	822.5	817.5	812.5	807.5	802.5	
885	880	875	870	865	880	855	850	845	840	835	830	825	820	100	810	805	800	
	+	2	to.	4	un.	ω	1	ω	o.	10	7	12	ta	4	5	9	11	
	(inches) (pst) (pst) (pst) (pst) (pst) (pst) (pst) (pst) (inches)	(inches) (psf) (psf) (psf) (psf) (psf) (psf) (psf) (psf) (inches) (15 156 119 7303 312 0.00 275 0 7890 0,5100 60	885 (psf) (inches) (inches) (sec. 110 275 156 119 7303 312 0.00 275 0 7890 0.5100 60 877.5 110 825 312 513 7303 312 0.00 825 0 8440 0.3405 60	885, (v.) (v.) (v.) (v.) (v.) (v.) (v.) (v.)	985 982.5 110 275 156 119 7303 312 0.00 275 0 7890 0.5100 60 877.5 110 825 312 513 7303 312 0.00 825 0 8840 0.3405 60 80 875 110 1375 312 1663 7303 312 0.00 1375 0 8890 0.2596 80 867.5 110 1375 312 1613 7303 312 0.00 1325 0 8540 0.2161 60	882.5 110 275 156 119 7303 312 0.00 275 0 7890 0.5100 60 60 877.5 110 825 312 513 7303 312 0.00 1375 0 8440 0.2405 60 80 877.5 110 1375 312 1663 7303 312 0.00 1375 0 8990 0.2596 80 80 875.5 110 1375 312 1613 7303 312 0.00 1425 0 5540 0.2161 60 860 860 860 865.5 110 2475 312 2163 7303 312 0.00 2475 0 10090 0.1873 60	985 982.5 110 275 156 119 7303 312 0.00 275 0 7890 0.5100 60 60 60 60 60 60 60 60 60 60 60 60 6	882.5 110 275 156 119 7303 312 0.00 275 0 7890 0.5100 60 875. 110 825 312 513 7303 312 0.00 825 0 8440 0.3405 60 877. 110 825 312 513 7303 312 0.00 1375 0 8990 0.2596 80 878. 110 1376 312 1613 7303 312 0.00 1325 0 1040 0.1873 60 889 882.5 110 2475 312 2163 7303 312 0.00 2475 0 10040 0.1862 80 889 857.5 110 3025 312 2713 7303 312 0.00 3025 0 10540 0.1662 80 885. 885. 110 3575 312 3283 7303 312 0.00 3575 0 11190 0.1499 80	Harry   Harr	882.5 110 275 156 119 7303 312 0.00 275 0 7890 0.5100 60  875 875 110 1375 312 513 7303 312 0.00 275 0 8440 0.5405 60  877 877.5 110 1375 312 1683 7303 312 0.00 1375 0 8990 0.2566 80  887 882.5 110 1375 312 1683 7303 312 0.00 1375 0 8990 0.2566 80  887 882.5 110 1375 312 1683 7303 312 0.00 1375 0 8990 0.2566 80  888 882.5 110 2475 312 2163 7303 312 0.00 2475 0 10080 0.1873 60  889 882.5 110 3025 312 2163 7303 312 0.00 3025 0 10640 0.1662 80  889 882.5 110 3025 312 2713 7303 312 0.00 3575 0 111740 0.1662 80  889 882.5 110 4425 312 3263 7303 312 0.00 4125 0 111740 0.1366 60	987.5         110         275         156         119         7303         312         0.00         275         0         7890         (psf)         (psf)	847         VVV         VVV <td>867.5         110         USFIT         U</td> <td>847         VM         VM</td> <td>887         110         275         156         119         7303         312         0.00         275         0.0         780         (psf)         (psf)</td> <td>882         110         275         166         119         7303         312         0.00         275         0.67         (psf)         (psf)</td> <td>887         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10</td> <td>840.7         11.0         17.5         15.6         11.6         17.5         15.6         11.6         17.5         15.6         11.6         17.5         15.6         11.6         17.5         15.6         11.6         17.5         12.6         17.5         15.6         11.6         73.0         31.2         0.00         27.5         0         45.0         0.51.0         0           87.6         87.5         11.0         12.5         31.2         51.3         730.3         31.2         0.00         27.5         0         64.0         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.40</td>	867.5         110         USFIT         U	847         VM         VM	887         110         275         156         119         7303         312         0.00         275         0.0         780         (psf)         (psf)	882         110         275         166         119         7303         312         0.00         275         0.67         (psf)         (psf)	887         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10         10	840.7         11.0         17.5         15.6         11.6         17.5         15.6         11.6         17.5         15.6         11.6         17.5         15.6         11.6         17.5         15.6         11.6         17.5         12.6         17.5         15.6         11.6         73.0         31.2         0.00         27.5         0         45.0         0.51.0         0           87.6         87.5         11.0         12.5         31.2         51.3         730.3         31.2         0.00         27.5         0         64.0         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.400         0.40

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

scv LX2006193 11/27/2006

Height 67 109 110 1.14 90 Summary of Parameters
Description Cc
Gypsum
Fly Ash 0.28

No.

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{{\sigma'}_{v_o}}{{\sigma'}_{v_o}}$$

Sublayer Compression	(menes)	5. 5.	7.3	6.1	5.3	4.7	4.2	3.8	3.5	3.3	3.1	2.9	2.7	2.8	
Sublayer Thickness	(miches)	8 9	09	09	09	90	09	09	09	09	09	09	09	09	
Change in Void Ratio (∆e)	0.5100	0.3405	0,2596	0.2161	0.1873	0.1662	0.1499	0.1368	0.1259	0.1168	0.1090	0.1022	0.0963	0.0911	
Final Vertical Effective Stress	7890	8440	8990	9540	10090	10640	11190	11740	12290	12840	13390	13940	14490	15040	
Final Pore Water Pressure	(iza)	0	0	0	0	0	0	0	0	0	0	0	0	0	
Final Total Final Pore Vertical Water Stress Pressure	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	5875	7425	
Added Pressure due to Embankment Loading	00.0	0.00	00:00	0.00	00'0	0.00	00'0	0.00	0.00	0.00	0.00	0.00	0.00	00.00	
Added Pressure due to 5' water	312	312	312	312	312	312	312	312	312	312	312	312	312	312	
Added Fill Pressure	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	
Initial Vertical Effective Stress	119	513	1063	1613	2163	2713	3263	3813	4363	4913	5463	6013	6563	7113	
Initial Pore Water Pressure	156	312	312	312	312	312	312	312	312	312	312	312	312	312	
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875	7425	
Total Unit Weight (pcf)	110	110	110	110	110	110	110	110	110	410	110	110	110	110	
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867.5	862.5	857.5	852.5	847.5	842.5	837.5	832.5	827.5	822.5	817.5	
Layer Top Elevation (ft)	8892	880	870	865	350	855	850	845	840	835	830	825	820	10	
Layer No.	þ	2	6	4	LO.	ø	7	60	o	10	41	12	13	14	

73.1

E.W. Brown Ash Pond Modification Results of Settlement Analysis Kentucky Utilities

11/27/2006

scv LX2006193

	Height	67	5
	¥.	100	110
	09		1.14
trameters	3		0.28
Summary of Para	Description	Gypsum	Fly Ash
	No.	1	2

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma'_{y_c}}{\sigma'_{y_o}}$$

Sublayer Compression (inches)	14.3	9.5	7.3	6.1	5.3	4.7	4.2	3.8	3.5	3.3	3.1	2.9	
Sublayer Thickness (inches)	90	09	09	90	09	09	09	09	09	09	09	09	
Change in Void Ratio (Δe)	0.5100	0.3405	0.2596	0.2161	0.1873	0.1662	0.1499	0.1368	0.1259	0.1168	0.1090	0.1022	
Final Vertical Effective Stress (psf)	7890	8440	8990	9540	10090	10640	11190	11740	12290	12840	13390	13940	
Final Pore Water Pressur e (psf)	0	0	0	0	0	0	0	0	0	0	0	0	
Final Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	
Added Pressur e due to Embank ment Loading (psf)	00:00	0.00	00.00	00.0	00'0	00.00	0.00	00.00	00.00	00'0	00.00	0.00	
Added Pressur e due to 5' water (psf)	312	312	312	312	312	312	312	312	312	312	312	312	
Added Fill Pressur e (psf)	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	
Initial Vertical Effective Stress (psf)	119	513	1063	1613	2163	2713	3263	3813	4363	4913	5463	6013	
Initial Pore Water Pressur e	156	312	312	312	312	312	312	312	312	312	312	312	
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	
Total Unit Weight (pcf)	110	110	110	110	110	110	110	110	110	110	110	110	
Void	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	
Layer Midpoint Elevatio n (ft)	882.5	877.5	872.5	867.5	862.5	857.5	852.5	847.5	842.5	837.5	832.5	827.5	
Layer Top Elevatio n (ft) 885	880	875	870	865	860	855	850	845	840	835	830	825	
Layer No.	÷	2	က	4	Ю	0	7	60	os	10	F	12	

Settlement Analysis Results Profile A-A (N-S).xls

scv LX2006193

	Height	579	
	×	109	110
	90		114
arameters	ပ္ပ		0.28
Summary of P	Description	Gypsum	Fly Ash
	No.	1.3	2

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma'_{\gamma'}}{\sigma'_{\gamma_o}}$$

									inches
Sublayer Compression (inches)	14.8	10.0	7.7	5.4	5.6	5.0	4.5	4.1	58.1
Sublayer Thickness (inches)	9	9	09	09	90	09	09	90	
Change in Void Ratio ( $\Delta e$ )	0.5280	0.3568	0.2743	0.2295	0.1995	0.1774	0.1601	0.1462	Total Settlement
Final Vertical Effective Stress (psf)	9149	9646	10145	10649	11155	11664	12176	12691	Total Se
Vater Pressure (psf)	0	0	0	0	0	0	0	0	
Final Total Final Pore Vertical Water Stress Pressure (psf) (psf)	275	825	1375	1925	2475	3025	3575	4125	
Added Pressure due to F Embankment Loading (psf)	1259.46	1205.53	1155.36	1108.63	1065.04	1024.33	986.26	950.61	
Added Pressure due to 5' water (psf)	312	312	312	312	312	312	312	312	
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	7303	7303	
Initial Vertical Effective Stress (psf)	119	513	1063	1613	2163	2713	3263	3813	
Initial Pore Water Pressure (psf)	156	312	312	312	312	312	312	312	
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	
Total Unit Weight (pcf)	110	110	110	110	110	110	110	110	
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867.5	862.5	857.5	852.5	847.5	
Layer Top Elevation (ft) 885	880	875	870	865	860	855	650	845	
Layer No.	-	2	e	4	ú	ω	1	60)	

952	Mid Point 7	882.5 69.5	877.5 74.5	872.5 79.5	867.5 84.5	862.5 89.5	857.5 94.5	852.5 99.5	847.5 104.5
	P.		0	0	0	0	0	0	0
	50	30.3	30.3	30.3	30.3	30.3	30.3	30.3	30.3
		109	109	109	109	109	109	109	109
Left	12	c 2	10	10	10	10	10	10	10
		0.41	0.39	0.36	0.34	0.33	0.31	0.30	0.28
	1	0 0	0	0	0	0	0	0	0
		1090	1090	1090	1090	1090	1090	1090	1090
		Δσz 142.72	134.09	126,40	119.52	113.32	107.71	102.61	97.97
		69.5	74.5	79.5	84.5	89.5	94.5	95.5	104.5
		B1 56.1	56.1	56.1	56.1	56.1	56.1	56.1	500
		B2 30.55	30.55	30.55	30.55	30.55	30.55	30.55	30.55
		7 109	109	109	109	109	109	109	300
	Right	표 우	10	10	10	10	10	10	Ç
		a.1 0.89	0.86	0.83	0.80	0.77	0.74	0.72	c c
		0.68	0.65	0.61	0.59	0.56	0.54	0.51	9
		1090	1090	1090	1090	1090	1090	1090	
		Δσz 1116.74	1071,44	1028.96	989.11	951.73	916.63	883.65	

Feet	4.3	4.6	4.9	5.4	5.9	6.5	8.4	6.7	6.1	5.7	4.8
Total Settlement	51.3	55.1	58.7	65.0	70.6	77.8	100.8	80.0	73.1	67.9	58.1
Location	A	В	0	D	Е	Н	9	Н	1	ſ	X

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

11/27/2006

scv LX2006193

nesults of Settlement Analysis		Height	7.0	5
Sememe		*	100	110
io stinsau		60		1.14
	arameters	33		0.28
	Summary of Parameters	Description	Gypsum	Fly Ash

Expression:

No.

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma' \psi}{\sigma' v_o}$$

	,
	007
48	
0.5363	Total Settlement
7835	Total Sa
0	
220	
0.00	
312	
7303	
56	
125	
220	
110	
1.14	
883.0	
188	
-	
	883.0 1.14 110 220 125 95 7303 312 0.00 220 0 7835

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

scv LX2006193 11/27/2006

> Height 67 109 110 1.14 00 0.28 Summary of Parameters Description Gypsum Fly Ash No.

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma'_{y'}}{\sigma'_{yo}}$$

						inches
Sublayer Compression (inches)	14.3	6,0	7.3	6.1	5.4	42.6
Sublayer Thickness (inches)	9	90	90	90	09	
Change in Void Ratio (Δe)	0.5100	0.3405	0.2596	0.2161	0.1931	otal Settlement
Final Vertical Effective Stress (psf)	7890	8440	8990	9540	9650	Total Se
Final Pore Water Pressure (psf)	0	0	0	0	D	
Final Total Final Pore Vertical Water Stress Pressure (psf) (psf)	275	825	1375	1925	2035	
Added Pressure due to Embankment Loading (psf)	0.00	0.00	0.00	0.00	0.00	
Added Pressure due to 5' water (psf)	312	312	312	312	312	
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	
Initial Vertical Effective Stress (psf)	119	513	1063	1613	1973	
Initial Pore Water Pressure (psf)	156	312	312	312	92	
Initial Total Vertical Stress (psf)	275	825	1375	1925	2035	
Total Unit Weight (pcf)	110	110	110	110	110	
Void Ratio	1,14	1.14	1,14	1.14	1.14	
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867.5	866.5	
Layer Top Elevation (ft) 885	880	875	870	865	868	
Layer No.	<del>*</del>	7	m	4	ເກ	

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

11/27/2006 scv LX2006193

Heinht	27	ò
**	100	110
60		1.14
33		0.28
lescription	ypsum	y Ash
	4A 08	ription

Expression:

$$\Delta H = H \left( \frac{C_e}{1 + e_o} \right) \log \frac{\sigma'_v}{\sigma'_{v_o}}$$

							inche
Sublayer Compression (inches)	143	e u	7.3	2 "	, e	2.8	45.2
Sublayer Thickness (inches)	9	09	09	09	09	98	
Change in Void Ratio (∆e)	0.5100	0.3405	0.2596	0.2161	0.1873	0.1671	otal Settlement
Final Vertical Effective Stress (psf)	7890	8440	8990	9540	10090	10530	Total Se
Final Total Final Pore Vertical Water Stress Pressure (Psf) (psf)	0	0	0	0	0	0	
Final Total Vertical Stress (psf)	275	825	1375	1925	2475	2915	
Added Pressure due to Embankment Loading (psf)	0.00	0.00	0.00	0.00	0.00	0.00	
Added Pressure due to 5' water (psf)	312	312	312	312	312	312	
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	
Initial Vertical Effective Stress (psf)	119	513	1063	1613	2163	2665	
Initial Pore Water Pressure (psf)	156	312	312	312	312	250	
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	2915	
Total Unit Weight (pcf)	110	110	110	110	110	110	
Void Ratio	1.14	1.14	1.14	1.14	1.14	1.14	
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867.5	862.5	858.5	
Layer Top Elevation (ft) 885	880	875	870	865	860	857	
Layer No.	-	73	m	4	co.	ω	

	Heinht	27.0	5
	Yŧ	109	110
	09		1.14
arameters	3		0.28
Summary of Pa	Description	Gypsum	Fly Ash
	No.	1	2

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma'_{sc}}{\sigma'_{so}}$$

Sublayer Compression (inches)	14.3	9.5	7.3	φ. 1-	5.3	4.7	4.2	3.8	3.5	3.3	
Sublayer Thickness (inches)	09	99	8	99	09	09	9	09	09	90	
Change in Void Ratio (∆e)	0.5100	0.3405	0.2596	0.2161	0,1873	0.1662	0.1499	0.1368	0.1259	0.1175	
Final Vertical Effective Stress (psf)	7890	8440	8990	9540	10090	10640	11190	11740	12290	12290	
Final Pore Water Pressure (psf)	0	D	0	0	0	0	0	0	o	0	
Final Total Final Pore Vertical Water Stress Pressure (psf) (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	4675	
Added Pressure due to Embankment Loading (psf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Added Pressure due to 5' water (psf)	312	312	312	312	312	312	312	312	312	312	
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	
Initial Vertical Effective Stress (psf)	119	513	1063	1613	2163	2713	3283	3813	4363	4675	
Initial Pore Water Pressure (psf)	156	312	312	312	312	312	312	312	312	0	
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	4675	
Total Unit Weight (pcf)	110	110	110	110	110	110	110	110	110	110	
Void Ratio	1.14	1,14	1.14	1,14	1.14	1.14	1.14	1,14	1.14	1.14	
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867.5	862.5	857.5	852.5	847.5	842.5	842.5	
Layer Top Elevation (ft) 885	880	875	870	865	860	855	850	845	840	845	
Layer No.	π	2	m	4	w	9	1	603	6	10	

inches

62.0

scv LX2006193

Summary of Parameters	ption Cc 80 Yr	100	
Sur	Descripti	Gypsum	Ch. Ant
	ó	-	

Expression:

$$\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma' y}{\sigma' v_o}$$

Sublayer Compression (inches)	14.3	9.5	7.3	6.1	5.3	4.7	4.2	3.8	3.5	3.3	3.1	2.9	2.7
Sublayer Thickness (inches)	09	09	09	09	09	09	09	09	09	09	09	90	9
Change in Void Ratio (∆e)	0.5100	0.3405	0.2596	0.2161	0.1873	0.1662	0.1499	0.1368	0.1259	0.1168	0.1090	0.1022	0.0963
Final Vertical Effective Stress (psf)	7890	8440	8990	9540	10090	10640	11190	11740	12290	12840	13390	13940	14490
Final Total Final Pore Vertical Water Stress Pressure (psf) (psf)	0	0	0	0	0	0	0	0	0	0	0	0	0
Final Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875
Added Pressure due to Embankment Loading (psf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Added Pressure due to 5' water (psf)	312	312	312	312	312	312	312	312	312	312	312	312	312
Added Fill Pressure (psf)	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303
Initial Vertical Effective Stress (psf)	139	513	1063	1613	2163	2713	3263	3813	4363	4913	5463	6013	6563
Initial Pore Water Pressure (psf)	156	312	312	312	312	312	312	312	312	312	312	312	312
Initial Total Vertical Stress (psf)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875
Total Unit Weight (pcf)	110	110	110	110	110	110	110	110	110	110	110	110	110
Void Ratio	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14
Layer Midpoint Elevation (ft)	882.5	877.5	872.5	867.5	862.5	857.5	852.5	847.5	842.5	837.5	832.5	827.5	822.5
Layer Top Elevation (ft) 885	880	875	870	865	860	855	850	845	840	835	830	825	820
Layer No.	~	2	63	4	ιο.	ø	7	œ	თ	10	7	12	5

inches

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E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

scv LX2006193 11/27/2006

> Helght 67 1,14 60 0.28 | Summary of Parameters |
> | Description | Cc |
> | Gypsum | Fly Ash | 0.28 Expression: No.

log  $\frac{\sigma'_{\gamma'}}{\sigma'_{\gamma''}}$  $\Delta H = H \left( \frac{C_c}{1 + e_o} \right)$ 

		5.																					
	į	Compression	(nucues)	14.3	20,	7.3	e e	r c	4.7	54	8	3.57	, e	e)	9.0	2.2	26	2.4	23	22	22	2.0	0,
	7	Thickness	(MICHES)	9	8	99	8	9	98	09	9	09	90	9	8	6	8	90	09	8	09	98	99
	Change in	(Ae)		0.5100	0.3405	0.2596	0.2161	0.1873	0.1662	0,1499	0.1368	0.1259	0.1168	0.1090	0.1022	0.0963	0,0911	0.0864	0.0822	0.0783	0.0749	7170.0	0.0682
	Vertical Vertical	Stress	(154)	7890	8440	8990	9540	10090	10540	11190	11740	12290	12840	13390	13940	14490	15040	15590	16140	16690	17240	17790	17955
	Final Pore	Pressure (nef)		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Final Total	Stress	Total Control	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875	7425	7975	8525	9075	9625	10175	10340
Added	Pressure due to to Embankment	Loading	1	0.00	0.00	0.00	0.00	0.00	00.00	00'00	0.00	00'0	0.00	00'0	00'0	00'0	0.00	0000	00'0	0.00	0.00	0.00	0.00
	Pressure due to 5'			312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312
	Added FIII			7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303
1	_ 0	Stress (psf)		119	513	1063	1613	2163	2713	3263	3813	4363	4913	5463	6013	6563	7113	7663	8213	8763	9313	5863	10246
Leiter	Pore	Pressure (psf)		156	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	25
Initial	Total Vertical	Stress (psf)		275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	5875	7425	7975	8525	9075	9625	10175	10340
	Total	Weight (pcf)		110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110
		Void Ratio	3	1.14	1.14	1.14	1.14	1,14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14
	Layer	Elevation (ft)		882.5	877.5	872.5	867.5	862.5	857.5	852.5	847.5	842,5	837.5	832.5	827,5	822.5	817,5	812.5	807.5	802.5	797.5	792.5	791.0
	Layer	Elevation (ft)	882	880	875	870	, to	980	658	850	845	840	835	830	825	820	12 2	810	805	800	795	790	792
	Layer	Š.		-	2	m	43	ıo	0	1	œ	on .	9	1	12	13	14	ů.	to.	17	9	ō.	20

14.

86.1

Sublayer Compression (inches)

Sublayer Thickness

14.3 9.5 7.3

0.5100 0.3405 63 4.7

0.2161 0,1873 0.1662 0.1499 0.1368 0.1259 0.1168

0.2596

69 3 2.8 2.6 2.4 23

0.0911

0.0864

0.0822 0.0783

0.1022 0.0963

0.1090

2.0 01 1.8

09 9 9

0.0717

0.0688 0,0859

Total Settlement

2.1

0.0749

Final
Vertical Change in
Effective Void Ratio S
Stress (Δe) T 18340 18725 10090 12840 13390 14490 15040 15590 17240 17790 7890 11740 13940 12290 Final Total Final Pore Vertical Water Pressure (Jsd) 0 Stress 11110 9625 (psd) 275 1375 1925 2475 825 3025 4125 4675 6875 7425 Added Pressure due to F Embankment Loading (bsd) 0.00 0.00 0.00 0.00 0.00 0.00 000 0000 0.00 0.00 0.00 0.00 00'0 0.00 0.00 000 000 Added Pressure due to 5' water (bst) 312 312 312 312 312 312 312 312 312 312 312 312 312 312 312 312 312 Added Fill 7303 (bst) 7303 7303 7303 Initial Vertical Effective Stress Height 67 10413 10892 6013 7113 8213 119 1063 2163 2713 3263 3813 4363 5463 6563 8763 9313 9863 Initial Pore Water Pressure 108 312 312 312 312 312 312 312 312 312 218 1.14 Initial Total Vertical Stress 90 10725 11110 10175 1375 2475 3575 4125 5775 6325 6875 7425 8525 9075 9625 825 Summary of Parameters
Description Cc
Gypsum
Fiy Ash 0.28  $\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma' \eta}{\sigma' v}$ Total Unit Weight 110 110 110 110 110 2 110 110 Layer Midpoint Elevation 784.0 882.5 877.5 872.5 867.5 857.5 852.5 847,5 842.5 837,5 832,5 827.5 822.5 817,5 812.5 807,5 802,5 797.5 792,5 787.5 E Layer Top Elevation 785 783 E Expression: Layer No. 20 2

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FMSM Engineers Lexington, KY

E.W. Brown Ash Pond Modification Kesults of Settlement Analysis

Height 57

Summary of Parameters
Description Cc
Gypsum
Fly Ash 0.28

 $\Delta H = H \left( \frac{C_c}{1 + e_c} \right) \log \frac{{\sigma^*}_{*c}}{{\sigma^*}_{*c}}$ 

Expression:

SCV LX2006193

11/27/2006

Initial         Pressure vertical         Pressure vertical         Pressure vertical         Incomplete         Incomplet
Initial   Initial   Pore   Vertical   Water   Effective   Pressure   Stress   Cips   119
titeal total
Layer Total Into Midpoint Unit Version Unit

FMSM Engineers Lexington, KY

E.W. Brown Ash Pond Modification Kentucky Utilities Results of Settlement Analysis

SCV LX2006193 11/27/2006

Height 67

0.28 1.14 Summary of Parameters
Description Cc
Gypsum
Fly Ash 0.28 1  $\Delta H = H \left( \frac{C_c}{1 + e_o} \right) \log \frac{\sigma' _{v}}{\sigma' _{v}}$ 

Expression:

Sublayer	Compression (inches)	20	10.0	7.1	2 %	i un	0.50	, st	17	M(2)	in in	1 173	ri ri	5	27	2.6	2.4	2.3	2.2	27	2.0	2.0			o (s	2.0
Sublayer	(Inches)	8	8	8	3 8	9	08	90	09	09	8	28	09	8	09	2	00	9	8	09	06	06	8	98	2	2
5 %	(PQ)	0.5273	0.3561	0.2737	0.2290	0.1990	0.1769	0.1597	0.1458	0,1343	0,1245	0.1163	0.1090	0.1026	0.0969	0.0919	0.0873	0.0832	0.0795	0.0761	0.0729	0.0700	0.0674	0.0649	0.0626	-
	(pst)	5606	9593	10096	10601	11110	11621	12135	12651	13169	13689	14211	14734	15259	15786	16313	16842	17372	17903	18435	18988	19502	20036	20572	21107	
	(pst)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	0	0	0	0	0	o	0	
Final Total Vertical	(pst)	275	828	1375	1925	2475	3025	3575	4125	4875	5225	5775	6325	6875	7425	7975	8525	9075	9625	10175	10725	11275	11825	12375	12925	
Added Pressure due to Embankment	(Jsd)	1204.60	1153.27	1105.52	1061,03	1019,54	980,77	944.51	910.54	878.68	348.76	320,52	794.11	769.13	745,54	723.25	702.15	562.17	663.22	645.22	628.12	611.85	595.36	581.59	567.50	
Added Pressure due to 5	(bsd)	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	
Added Fill	(bst)	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	7303	
Vertical Vertical Effective Stress	(Jsd)	119	513	1063	1613	2163	2713	3263	3813	4363	4913	5463	6013	6563	7113	7663	8213	8763	9313	9863	10413	10963	11513	12063	12613	
Initial Pore Water Pressure	(jsd)	156	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	312	342	312	312	312	
Initial Total Vertical Stress	(bst)	275	825	1375	1925	2475	3025	3575	4125	4675	5225	5775	6325	6875	7425	7975	8525	9075	3625	10175	10725	11275	11825	12375	12925	
Total Unit Weight	(bd)	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110	
Layer Midpoint Elevation	Œ	882.5	877.5	872.5	367.5	862.5	857,5	852.5	847.5	842.5	837.5	832.5	827.5	822.5	817.5	812.5	807.5	802.5	797.5	792.5	787.5	782.5	2777	772.5	767.5	
Layer Top Elevation	(H)	380	375	970	2 2	860	9255	850	845	840	8335	830	825	820	515	810	805	800	795	790	765	780	775	2 22	2 4	3
Layer No.		-	2	m	4	us	0	1	03	g)	10	7	12	12	4	10	15	17	00	2	20	5	22	23	24	

XIS	
E-W	
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d Point 882.5		877.5	872.5	867.5	862.5	857.5	852.5	847.5	842.5	837.5	832.5	827.5	822.5	817.5	812.5	807.5	802.5	797.5	792.5	787.5	782.5	2777	772.5	767.5
58.5		74.5	79.5	84.5	89.5	94.5	99.5	104.5	109.5	114.5	119.5	124.5	129.5	134.5	139.5	144.5	149.5	154.5	159.5	164.5	169.5	174.5	179.5	1845
0	,	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	c
31.4		31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31,4	31.4	31.4	2. 40
Y 109	2	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	007
# Ç	2	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	
0.42	0.42	0.40	0.38	0.36	0.34	0.32	0.31	0.29	0.28	0.27	0.26	0.25	0.24	0.23	0.22	0.21	0.21	0.20	0.19	0.19	0.18	0.18	0.17	
α2	D	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	16
960	1080	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	
Aoz	147.31	138.47	130.58	123.50	117.13	111.36	106.11	101.33	96.94	92.91	89.20	85.76	82.58	79.61	76.86	74.28	71.87	69.60	67.48	65.47	63.59	61.80	60.12	
2 0	69.5	74.5	79.5	84.5	89.5	94.5	99.5	104.5	109.5	114.5	119.5	124.5	129.5	134.5	139.5	144.5	149.5	154.5	159.5	164.5	169.5	174.5	179.5	
84	22	53	133	25	55	33	53	55	53	55	55	B	83	93	92	33	23	B	33	55	99	32	32	
82	34	34	34	34	8	34	34	25	35	34	35	8	35	34	34	34	34	34	34	34	34	34	34	
>	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	109	
I	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	
a.	0.91	0.87	0.84	0.81	0.78	0.76	0.73	0.71	0.68	99.0	0.64	0.62	09.0	0.58	0.57	0.55	0.54	0.52	0.51	0.50	0.48	0.47	0.46	
g2	29'0	0.64	0.61	0.58	0.55	0.53	0.50	0.48	0.47	0.45	0.43	0.42	0.40	0.39	0.38	0.36	0.35	0.34	0.33	0.32	0.31	0.31	0.30	
ob	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	1090	
Aoz	1057.29	1014.80	974.94	937.53	902.41	869.41	838.40	809.22	781.74	755.85	731.42	708.35	686.55	665.93	646.39	627.87	610.30	593.61	577.75	562.65	548.27	534.56	521.47	

al nent t)					1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1				
Total Settlement (feet)	1.0	3.5	3.8	5.2	5.9	7.2	7.3	7.5	8.2
Total Settlement (inches)	12.0	42.6	45.2	62.0	70.6	86.1	87.9	89.7	98.3
Location	A	В	O	D	Е	Н	5	Н	1

Appendix G
SITES Digital Data (CD)

### PLANS FOR CONSTRUCTION

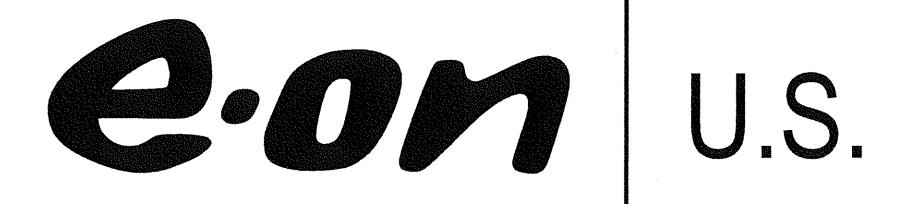
# MAIN ASH POND - STARTER DIKE ASH POND EXTENSION PROJECT E.W. BROWN GENERATING STATION

MERCER COUNTY, KENTUCKY

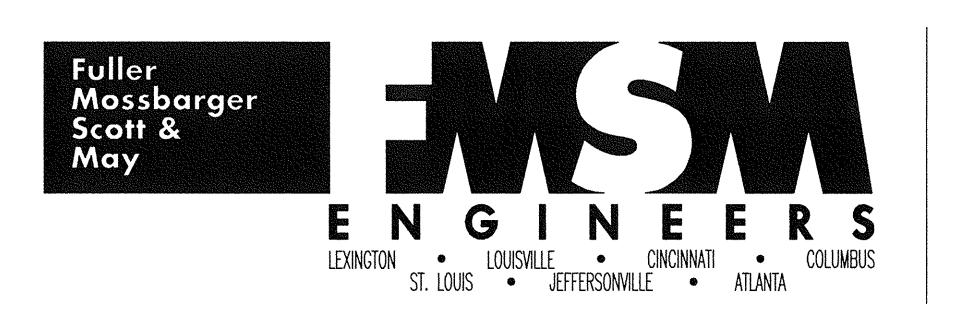
PREPARED FOR

# KENTUCKY UTILITIES COMPANY

LEXINGTON, KENTUCKY



PREPARED BY



SCOTT AND MAY ENGINEERS, INC.

1409 N. Forbes Rd. Lexington, Kentucky 40511–2050

HAZARD CLASSIFICATION HIGH HAZARD, CLASS "C" D.O.W. INVENTORY NO. KY 737

REVISIONS

Revisions

Revisions

Revisions

Revisions

Revisions

Revisions

Revisions

Revision Mode

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Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN

Drawn: TJ

Date: NOVEMBER, 2007

Chacked: VIS/DAB

Approved:

Revision Mode

Location and Unit:

E.W. BROWN GENERATING STATION

Kentucky

Utilities

Company

Approved:

JOB NO. JOB NO. JOB NO. JOB NO. JOB NO. Drawing No:

BRO-C-00800

VICINITY MAP

1 0 2 4 MILES

GRAPHIC SCALE

### INDEX OF SHEETS AND REVISIONS

				INDEX OF SHEET	S			
Sheet No.	Description	Revision No.	Sheet No.	Description	Revision No.	Sheet No.	Description	Revision No.
neral Arrangement			Embankment & Line	r		Site Characterization		
-C-00800	Cover Sheet	T A	BRO-C-00826	Surface Drainage Layout and Details	A	BR0-C-00851	Hydraulic and Hydrologic Data — Main Pond	Ā
)-C-00801	Index of Sheets and Revisions	A	BR0-C-00827	Surface Drainage Layout and Details	A	BRO-C-00852 [MAP 1]	Plan View — Boring Layouts	A
-C-00802	General Notes	A	BR0-C-00828	West Side Sump Layout and Plan View	**************************************		Plan View — Boring Layouts	A
	Plan View - Project Setting Sheet (1"=300')	A	BR0-C-00829	NOT USED		BRO-C-00854 [MAP 3]	Plan View — Boring Layouts	A
	Plan View - Map Key (1"=300')	A	BR0-C-00830	Typical Sections — Main Ash Pond — Starter Dike	Α	BRO-C-00855 [MAP 4]	Plan View — Boring Layouts	Α
	Starter Dike Overview Plan (1"=200')	A	BR0-C-00831	Typical Sections - Main Ash Pond - Starter Dike	Α	BRO-C-00856 [MAP 5]	Plan View Boring Layouts	A
-C-00806	Final Embankment Overview Plan (1"=200')	A	BR0-C-00832	Typical Sections — Main Ash Pond — Starter Dike	A	BR0-C-00857	Logs of Borings – Main Ash Pond Perimeter (RD)	A
- <i>C00807</i> [MAP 1]	Existing Conditions and Baseline Layout (1"=100')	A	BR0-C-00833	Typical Sections — Main Ash Pond — Starter Dike	A	BR0-C-00858	Logs of Borings - Main Pond Borrow Area (MP)	A
	Existing Conditions and Baseline Layout (1"=100')	A	BR0-C-00834	Profile — Main Ash Pond — Starter Dike	Α	BR0-C-00859	Logs of Borings — Main Ash Pond Perimeter (ASH)	A
C-00809 [MAP 3]	Existing Conditions and Baseline Layout (1"=100')	A	BR0-C-00835	Details — Foundation Treatment	Ā	BR0-C-00860	Logs of Borings — Houp Property Borrow Area (BA)	A
	Existing Conditions and Baseline Layout (1"=100')	A	BR0-C-00836	Details – Liner System	A	BR0-C-00861	Logs of Borings — Houp Property Borrow Area (BA)	A
	Existing Conditions and Baseline Layout (1"=100')	A	BR0-C-00837	NOT USED		BR0-C-00862	Logs of Borings — Houp Property Borrow Area (BA)	A
	Existing Conditions and Baseline Layout (1"=100')	A	BR0-C-00838	NOT USED		BR0-C-00863	Logs of Borings — Houp Property Borrow Area (BA)	A
	Starter Dike Construction Plan (1"=100')	<u> </u>				BR0-C-00864	Logs of Borings — Riser Structure (RS)	A
C-00814 [MAP 2]	Starter Dike Construction Plan (1"=100')	<u>A</u>				BR0-C-00865	Stability Analyses — Main Ash Pond — Section A	A
	Starter Dike Construction Plan (1"=100')	A				BRO-C-00866	Stability Analyses - Main Ash Pond - Section B	A
	Starter Dike Construction Plan (1"=100')					BR0-C-00867	Stability Analyses — Main Ash Pond — Section C	A
		A				BR0-C-00868	NOT USED	
C-00818	Other Contractor Limits NOT USED					BR0-C-00869	NOT USED	
-C—00819 -C—00820	NOT USED					Embankment Cross \$	Sections	
-C-00821	NOT USED					BR0-C-00870	Embankment Cross Sections — Sta. 500+00 to Sta. 502+50	A
<del></del>						BR0-C-00871	Embankment Cross Sections — Sta. 503+00 to Sta. 505+50	
iment Control / G	eneral Items					BR0-C-00872	Embankment Cross Sections — Sta. 506+00 to Sta. 508+50	
-C-00822	Details — Sediment and Erosion Control	T				BR0-C-00873	Embankment Cross Sections - Sta. 509+00 to Sta. 511+50	A
	Piezometer Details	A	<u> </u>			BR0-C-00874	Embankment Cross Sections - Sta. 512+00 to Sta. 514+50	A
-C-00824	NOT USED					BR0-C-00875	Embankment Cross Sections - Sta. 515+00 to Sta. 517+50	A
C-00825	NOT USED					BR0-C-00876	Embankment Cross Sections - Sta. 518+00 to Sta. 520+50	A
						BR0-C-00877	Embankment Cross Sections — Sta. 521+00 to Sta. 523+50	A
			California			BR0-C-00878	Embankment Cross Sections - Sta. 524+00 to Sta. 526+50	A
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			BR0-C-00839	Profile/Details — Principal Spillway Pipe — Main Ash Pond	A	BR0-C-00880	Embankment Cross Sections — Sta. 530+00 to Sta. 532+50	A
		State of Proceed And Annual Control of Contr	BR0C00840	Profile/Details — Secondary Spillway Pipe — Main Ash Pond	A	BR0-C-00881	Embankment Cross Sections — Sta. 533+00 to Sta. 535+50	A
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			BR0-C-00842	Details — Riser Structure	A	BR0-C-00883	Embankment Cross Sections — Sta. 539+00 to Sta. 541+50	A
			BR0-C-00843	Details — Riser Structure	<u> </u>	BR0-C-00884	Embankment Cross Sections — Sta. 542+00 to Sta. 544+50	Α
444-is-4			BR0-C-00844	Details - Riser Structure	A	BR0-C-00885	Embankment Cross Sections — Sta. 545+00 to Sta. 547+50	A
			BR0-C-00845	Details - Riser Structure	A	BR0-C-00886	Embankment Cross Sections — Sta. 548+00 to Sta. 550+50	A
			BRO-C-00846	Profile - Access Bridge Pier	<u>A</u>	BR0-C-00887	Embankment Cross Sections - Sta. 551+00 to Sta. 553+50	<u>A</u>
			BRO-C-00847	Details — Access Bridge Pier	A	BR0-C-00888	Embankment Cross Sections — Sta. 554+00 to Sta. 556+50	A
			BRO-C-00848	NOT USED		BR0-C-00889	Embankment Cross Sections - Sta. 557+00 to Sta. 559+50	A
			BR0-C-00849	NOT USED		BR0-C-00890	Embankment Cross Sections — Sta. 560+00 to Sta. 562+50	<u>A</u>
			BR0-C-00850	NOT USED		BR0-C-00891	Embankment Cross Sections — Sta. 563+00 to Sta. 565+50	A
						BR0-C-00892	Embankment Cross Sections - Sta. 566+00 to Sta. 568+50	A
						BR0-C-00893	Embankment Cross Sections — Sta. 569+00 to Sta. 571+50	
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							Embankment Cross Sections — Sta. 5/8+00 to Sta. 580+50   Embankment Cross Sections — Sta. 581+00 to Sta. 582+50	
						BR0-C-00897	Embankment Gross Sections - Sta. 581+00 to Sta. 582+50	<u> </u>

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Revision No.	Date	Sheet Nos.	Description	Revision No.	Date	Sheet Nos.	Description	Revision No.	Date	Sheet Nos.	Description
A	04-23-08	All Sheets	Released For Construction								
A	04-23-08	BR0-C-00802 &	Revised/Added Notes						tion of the state of the control of the control of the control of the state of the	s and house a service man at an advanta of this color in the order of a service of a service and a service or a service of	
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MAP 1 MAP 2 MAP 3
MAP 4 MAP 5
MAP 6

- SUPPLEMENTAL GENERAL NOTE: Borrow source embankment material will be utilized for the Starter Dike project. Therefore, processed gypsum embankment material previously indicated in the plan set has been replaced with Type IIa-24 rock embankment material to build the Starter ( Dike embankment. Any reference to gypsum embankment material in these plans shall be interpreted as using Type IIa-24 rock embankment material with the exception of a 12 inch layer of bottom ash will be placed to cover and protect geo-reinforcing material in the working platform area. Any changes to the embankment material source used by the Contractor during the course of the project will be as directed / approved by the Owner's Representative. A plus (+) symbol appears by a General Note to identify where Type IIa-24 rock embankment material or bottom ash is to be used in place of the previously indicated gypsum embankment material.
- The purpose of the project is to construct the Starter Dike phase of the Main Ash Pond (+)expansion at the E.W. Brown Generating Station by January 2011. During this contract, the Main Ash Pond will be non-operational with all process flows diverted to the recently constructed Auxiliary Ash Pond. Excavation and pumping operations will be required to drain the low-lying areas of the pond before the existing hydraulically placed ash surface can be leveled and regraded prior to beginning construction of the <del>gypeum</del> bottom ash/Type Ila-24 working platform and Starter Dike embankment. Gypsum byproduct from the dew material. The Starter Dike embankment section also includes clay and crushed limestone embankment zones. A flexible membrane liner (FML) will be installed on the pond bottom and along the upstream slopes of the Starter Dike embankment. Filter fabrics will also be used to separate <del>aypeum</del> bottom ash/Type IIa—24 and clay embankment materials and to provide protection for the FML. A new spillway riser structure, bridge pier, pedestrian bridge, and scaffold stairway will be constructed during the Starter Dike phase. One span of the pedestrian bridge from the pier to the riser will be constructed with this contract. A second bridge span will be installed in a future project. Storm water runoff from the working platform will be directed to a storm water collection system and pumped to the operating Auxiliary Ash Pond. The completed Starter Dike will provide storage for approximately three years of sluiced fly ash. During future expansion phases, the embankment will be raised with downstream embankment overlays and the FML will be extended up the upstream face of the raised embankment
- 2. Topographic information was obtained from two aerial surveys performed by L. Robert Kimball & Associates of Ebensburg, Pennsylvania. The clip line between these two surveys is shown on Sheets 803 through 812. Topographic mapping of the existing Main Ash Pond embankment crest above elevation 880' was obtained from a topographic survey performed by BA Engineers. Hydrographic information (bottom of pond contours) was developed by FMSM Engineers on October 25, 2006 using a mapping grade GPS locator device and sonar equipment.

- 3. The property line information shown herein is a general approximation of property limits reconstructed from property deeds. The property lines have not been field surveyed and are not accurate for conveyance of property. The property lines should only be considered a aeneral representation.
- 4. All existing barns, houses, foundations and utilities within borrow areas shall be removed, when needed, by the OWNER.
- 5. Sediment control fences and rock check dams shall be installed along the downstream toe of all embankment phases under construction and around all borrow areas as shown on the drawings and as directed by the OWNER'S REPRESENTATIVE.
- 6. The CONTRACTOR shall not disturb existing power lines and shall not excavate around existing towers and poles within the following distances from the center of the structure unless otherwise noted or directed. Slopes shall not exceed 2:1 (H: V) and the CONTRACTOR shall maintain access to all transmission structures.

<u>Description</u>	Radius (ft.)
Guyed Single Steel Pole	120
Large Lattice Transmission Tower	100
Guyed Single Wood Pole	<i>75</i>
Double Pole Wood Structure	60
Single Pole Structure (Wood or Metal)	<i>50</i>
Guying Anchor	30

A ten (10) feet horizontal buffer zone must be maintained between the construction traffic and the power poles/towers at all times.

7. In addition to the horizontal clearances, the following vertical and transit clearances shall be observed for transmission lines.

<u>ertical Consti</u>	<u>ruction Clearances</u>
<u>Description</u>	Clearance (ft.)
69 kV	12
138 kV	16
345 kV	<i>25</i>

The above distances shall be measured from the highest point on the equipment to the lowest point on the line within the working area of the equipment.

- 8. The CONTRACTOR shall conduct his operations within the construction limits indicated on the drawings. Where temporary limits are shown, the CONTRACTOR shall enter those areas only for the duration of time required to complete his Work. The OWNER will have other contractors performing work within these defined project construction limits and at nearby areas during the time of this work. CONTRACTOR shall coordinate with other contractors in the execution of the work and shall accommodate access route revisions and minor delays, share work areas and other coordination efforts without additional cost to the OWNER.
- 9. The haul road to the Houp Property (Borrow Area Nos. 2 and 3) crosses the primary entrance road to the E.W. Brown Generating Station. Plant deliveries, generating station personnel, construction deliverables, and construction personnel for the scrubber construction will be using the plant entrance continually, therefore access must be maintained at all times. The design of this road shall be the responsibility of the CONTRACTOR and limited to the area shown on the site drawings. A flagman will be required when construction equipment crosses the plant access road and Curdsville Road.
- 10. The CONTRACTOR is responsible for coordination with the railroad company in constructing track crossings and in the coordination of rail deliveries. The CONTRACTOR shall expect that the rail crossing will be blocked at times during delivery and pick-up of rail cars. Schedules shall be adjusted accordingly and no additional costs or time shall be A considered
- \_\_\_\_\_ 11. Borrow excavations shall be limited to those locations within Borrow Area Nos. 1. 2 and 3 (+) with appropriate offsets from property lines, wetlands, transmission towers and other utilities as shown on the drawings. Borrow Area No. 1, located within the Main Ash Pond, consists of bottom ash material deposits previously sluiced over time to this area of the pond. The bottom ash is to be excavated and stockpiled for later use as a ballast material. Borrow Area No.1 shall be the only source of bottom ash for the project. Borrow Area No.1 is underlain by an existing FML at approximately Elevation 880' between Sta. 517+00 and Sta. 535+00. The FML extends approximately 180 feet into the ash pond from the existing embankment centerline as shown on the drawings. The FML shall not be disturbed and bottom ash borrow excavations shall not extend below elevation 880°. No bottom ash material is to be wasted without the special written approval of the OWNER'S REPRESENTATIVE. Borrow Area Nos. 2 and 3 are located offsite across Curdsville Road at the former Houp property as shown on the drawings. Borrow Areas Nos. 2 and 3 shall be the only sources of rock and clay embankment material unless otherwise directed by the OWNER'S REPRESENTATIVE. No clay embankment material is to be wasted without the special ( written approval of the OWNER'S REPRESENTATIVE.
- 12. Delivered materials for incorporation into the work shall be temporarily stored in areas as indicated on the drawings and/or as selected by the CONTRACTOR and approved by the OWNER'S REPRESENTATIVE. The CONTRACTOR parking and office areas shall not be used for temporary storage.

## GENERAL NOTES

- 13. The existing Auxiliary Ash Pond must remain in operation at all times. The CONTRACTOR shall maintain the ÓWNER'S access to all portions of the Auxiliary Ash Pond and the KPDES discharge/monitoring point during the project.
- 14. The existing riser may be used to dewater the Main Ash Pond prior to ash surface grading operations. If at any time effluent monitoring due to pond dewatering exceeds facility discharge criteria, a temporary siphon or pump system shall be installed according to the specifications to drain to the Auxiliary Ash Pond.
- 15. The new riser structure, bridge pier, bridge and spillway pipes must be constructed before the existing riser structure is partially removed, grouted and abandoned.
- 16. All existing hillside areas around the perimeter of the Main Ash Pond designated on the plans to receive insitu foundation treatment prior to embankment and FML construction shall be cleared, grubbed, and stripped of all vegetation. The final excavation depth and extent of foundation treatment shall be as determined by the OWNER'S REPRESENTATIVE during construction. When insitu treatments are completed, all disturbed areas shall be uniformly graded to drain towards the Main Ash Pond with a minimum two (2) percent arade. Insitu treatment along the existing hillside greas along the perimeter of the Main Ash Pond shall be extended below the perimeter of the ultimate regraded ash deposits to Elevation 878.5' (or 5 feet below any revised ash grading elevation) as shown on the drawings. The existing ash deposits around the perimeter of the pond shall be excavated to expose original ground prior to performing insitu treatment. Excavated slopes in the A ash deposits shall not exceed 2H:1V.
- 17. Completed insitu foundation treatment areas designated to be covered with FML shall be (+) backfilled with a minimum of two feet of compacted clay embankment and graded to drain towards the Main Ash Pond prior to FML installation. Completed insitu treatment areas -construction. In no case shall FML or gypsum embankment be placed directly upon
- 18. All cleared topsoil material shall be stockpiled in the areas designated on the drawings and used for final dressing. Topsoil stripped from borrow areas shall be used for final dressing within the respective borrow area.
- 19. Stockpile and waste areas shall be graded to maintain positive drainage at all times. The side slopes shall have a 2:1 maximum slope. The top shall have a two (2) percent minimum slope. Segregate materials as directed by the OWNER'S REPRESENTATIVE. Final grading and revegetation of these areas shall be performed under this contract.
- 20. All soft and saturated materials within the embankment limits shall be removed as directed by the OWNER'S REPRESENTATIVE.
- 21. Without regard to the materials encountered, all excavation shall be unclassified, unless noted otherwise.
- 22. The CONTRACTOR shall complete pre-blast surveys of all structures and improvements within 2,000 feet of the limits of proposed blasting activities which are outside the E.W. Brown Generating Station property line. The survey shall be completed at least 15 days prior to planned blasting. Blasting work will be permitted upon the approval of the pre-blast survey by the OWNER'S REPRESENTATIVE.
- $\bar{c}$ 23. Gypsum embankment material shall consist of ayosum, an FGD byproduct of the scrubber (+) unit, obtained from the Gypsum Dewatering Facility. This material shall be free of organic materials and deleterious substances. No gypsum material is to be wasted without the approval of the OWNER'S REPRESENTATIVE.
- 24. The Cypsum Dewatering Facility will continuously process and stockpile dewatered gypsum (+) during normal plant operations, and stockpile areas at the dewatering facility may be limited to three or four days of gypsum production. The CONTRACTOR shall load and haul gypsum material from the stockpile area on a nearly daily basis in a manner to allow the OWNER to continuously stockpile gypsum within the limited stockpile areas at the Gypsum Dewatering Facility. For bidding purposes, the CONTRACTOR shall assume that stockpiled gypsum at the Gypsum Dewatering Facility will meet the material and moisture requirements of the specifications for gypsum embankment material. It shall be assumed that gypsum material will be produced, dewatered, and stockpiled at the Gypsum Dewatering Facility at a rate of approximately 1,000 to 1,500 cubic yards per day, and SONTRACTOR shall subsequently plan embankment construction sequences accordingly. The CONTRACTOR shall be responsible for all gypsum handling, stockpiling, placement and moisture requirements after the gypsum has been produced at the Gypsum Dewatering Facility.
- 25. Clay embankment material shall consist of plastic clay materials, free of organic material, which classify as CH. CL, MH. ML, CL-ML, SC or SM-SC according to the Unified Soil Classification System. The maximum permissible dimension of stones or rocks shall be three (3) inches. All clay embankment material shall consist of Soils 5 and 6 from Borrow Area Nos. 2 and 3. Under no circumstances shall clay embankment material be wasted unless approved by the OWNER'S REPRESENTATIVE.
- 26. Bottom Ash embankment material shall consist of coal combustion bottom ash from Main Ash Pond Borrow Area No. 1.
- 27. Ash surface grading embankment material shall consist of coal combustion fly ash excavated from the Main Ash Pond surface. This material shall be free of organic materials and deleterious substances.
- 28. No. 68 stone and No. 2 stone embankment material shall consist of crushed rock or gravel meeting the requirements as given in Section 805 of the Kentucky Department of Highways "Standard Specifications for Road and Bridge Construction", current edition. No. 57 stone (modified) embankment material shall also meet the above requirements, except the maximum size of the aggregate shall be (1) one inch. Other than where required for the Starter Dike liner system, standard No. 57 stone can be used elsewhere on the project as stated in the specifications.
- 29. The CONTRACTOR shall construct the embankment in the zones and with the types of A materials required in the Contract Documents.
- 30. Under no circumstances shall rock be substituted for clay <del>or gypsum embankment</del> materials.
- 31 The foundation of the embankments that consist of soil outside the limits of the hydraulicall placed ash shall be proof-rolled a minimum of one pass with a fully loaded Volvo A40 dump truck, or equivalent, and four passes with a vibratory smooth drum roller. Any areas of pumping shall be removed or stabilized as directed by the OWNER'S REPRESENTATIVE.
- 32. After the Main Ash Pond is taken out of service and dewatered, the remaining hydraulically placed ash deposits shall be regraded and leveled to approximately Elevation 883.5'. This elevation was estimated using a hydrographic survey performed in October 2006 and expected production rates of fly ash and bottom ash byproducts between October 2006 and January 2008. The final regrade elevation will be determined by the OWNER'S REPRESENTATIVE based on the actual byproduct ash volumes and the final dewatered ash surface. It is intended that the area within the pond be graded to a level surface using all the ash material present which is not required for liner system ballast and the working platform (bottom ash).
- 33. A working platform consisting of gypsum bottom ash/Type IIa-24 embankment and biaxial (+) geogrid reinforcement shall be constructed upon all ash deposits located within the footprint of the final <del>aypoum</del> embankment configuration shown on Sheet 806. The ash deposits in the **J** area of the working platform shall be regraded to approximately Elevation 885.0'.
- 34. A stormwater collection system shall be installed within the working platform area between the existing embankment and the Starter Dike as shown on the Drawings. Two wet well duplex pump stations shall be installed to pump collected stormwater to the Auxiliary Ash

- 35. The CONTRACTOR shall be responsible for dust control management from all ash surfaces during the ash dewatering, regrading, and embankment construction operations for the A duration of the project in accordance with the specifications.
- \_\_\_\_\_ 36. The embankments shall be constructed in approximate horizontal lifts extending the entire (+) length and width of the embankment. For a waiver to be granted, the CONTRACTOR shall submit a plan to the OWNER'S REPRESENTATIVE for review and approval containing a method that will allow proper utilization of embankment material for less than full width construction. The gypeum embankment shall be constructed to full height before FML
- \$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$\$ A 37. Gypsum embankment material shall be placed in layers not to exceed eight (8) inches in (+) uncompacted thickness. Moisture control and compaction shall be in accordance with the
- 38. Clay embankment material shall be placed in layers not to exceed eight (8) inches in uncompacted thickness. Moisture control and compaction shall be in accordance with the
- 39. Bottom ash (liner ballast/cover) shall be placed in a single lift, eighteen (18) inches thick over the FML and Type II filter fabric across the regraded ash deposits. Moisture control will not be required for bottom ash placement. The single lift shall be bladed with a low ground pressure dozer with a maximum static weight of 46,000 pounds and a maximum contact pressure of 4.9 pounds per square inch (psi). Only broad turns will be allowed with low around pressure equipment anytime material is placed over the FML. A single 12 inch lift of
- 40. A three (3) foot—thick layer of bottom ash shall be placed over the FML along all truck haul roads, material handling areas, and equipment turning areas during bottom ash (liner ballast/cover) placement within the Main Ash Pond.

bottom ash shall be applied in the working platform above the geo-reinforcing material.

- 41. Ash surface grading embankment material shall be placed in layers not to exceed eighteen (18) inches in uncompacted thickness. Moisture control and compaction shall be in accordance with the specifications.
- 42. Modified No. 57 stone, with a maximum aggregate size of one—inch, shall be placed in a single, minimum 12-inch thick lift along the upstream face of the Starter Dike embankment over the FML and Type II filter fabric. The single lift shall be pushed onto the slope with a low ground pressure equipment with a maximum static weight of 46,000 pounds and a maximum contact pressure of 4.9 pounds per square inch (psi). Stone placement shall begin at the base of the slope and proceed up the slope. Low ground pressure equipment shall only traverse up and down the slope with no turns or sudden starts and/or stops.
- 43. No. 57 stone, No. 68 stone, No. 2 stone, and No 57 stone (modified) embankment materials shall be placed in full lifts according to the drawings and specifications. Moisture control will not be required for these materials.
- 44. If the surface of the prepared foundation or the rolled surface of any layer of the compacted earth fill is too dry or smooth to bond properly with the layer of material to be placed thereon, it shall be moistened and/or worked with a harrow, scarifier, or other suitable equipment, in an approved manner to a sufficient depth to provide a satisfactory bonding surface before the next succeeding layer of material is placed. If the rolled surface of any layer of the fill in place is too wet for proper compaction of the layer of material to be placed thereon, it shall be removed, allowed to dry, or be worked with a harrow, scarifier, or other suitable equipment to reduce the water content to the required amount, and then it shall be recompacted before the next succeeding layer of material is placed.
- 45. During construction, the top surface of all <del>gypeum</del> embankment fill areas shall be kept sloped (+) at a minimum two (2) percent grade. The gypcum surface within the working platform shall be araded to a minimum slope of one and one-half (1-1/2) percent to promote positive stormwater drainage to the stormwater collection system.
- 46. All pipe shall meet the requirements of the project specifications. Pipe shall not be rolled, dropped, or thrown into the trench. Pipe that is not in true alignment or which shows abnormal settlement after placement, shall be removed and re-laid.
- 47. Under no circumstances shall construction equipment travel over pipe installations until at least  $\Lambda$  two (2) feet of compacted backfill has been placed above the top of the pipe bedding.

			Compa	ction Requireme	ents	
	Max. Lift		Optimum	Vibratory	Self Propelled	Loadea
Material	Thickness		Moisture Content	Smooth Wheel	Static	Rock
Туре	(inches)	Proctor	Requirement	Roller	Roller	Truck
Clay	8	95%	-2% to +4%	Auto- Union	As Needed	
-Cypsum	8	95%	2% to 14%	If Needed	As Needed	
Fly Ash	8/18	92%	-4% to +4%	If Needed	As Needed	
Bottom Ash	18		Spread with Lo	ow Ground Pres	sure Dozer	
Type IIa-24	30			6 Passes		1 Pass
Modified No. 57 Stone	12		Spread with L	ow Ground Pres	ssure Dozer	
No. 2 Stone	12		Spread with Lo	ow Ground Pres	ssure Dozer	

- \* See other general notes and specifications for more specific requirements. 49. Type I and Type II geotextile filter fabric shall be a non-woven, polyester or polypropylene fabric meeting the requirements of the specifications.
- 50. Flexible membrane liner shall consist of co-extruded white (top) and black (bottom) textured 60 mil (total thickness) linear low density polyethylene (LLDPE) liner meeting the requirements of the specifications.
- 51. Base course for the access road and embankment surfacing shall consist of nine (9) inches of No. 2 stone. Base course stone shall conform to Section 805 of the Kentucky Department of Highways "Standard Specifications for Road and Bridge Construction", current edition.
- 52. Top course for the access road and embankment surfacing shall consist of nine (9) inches of No. 610 stone, and shall conform to Section 805 of the Kentucky Department of Highways "Specifications for Road and Bridge Construction", current edition.
- 53. Instrumentation (piezometers) shall be installed in accordance with the drawings and specifications. The CONTRACTOR shall conduct his operations in such a manner that the instruments and associated equipment will not be damaged by later construction work during this contract. Suitable markers, guard posts, or other approved means shall be provided by the CONTRACTOR to identify instrumented areas and readout terminals. Any damage occurring during the period of this contract shall be repaired at no additional cost to the OWNER. All monitoring will be conducted by the OWNER'S REPRESENTATIVE.
- 54. Final grading, fertilization, seeding and mulching of all disturbed areas shall be completed as soon as practical after completion of such respective portions of the project. Borrow areas shall be reseeded as each segment has been exhausted of borrow material.

55. Concrete:

Class A Concrete — f'c

4,500 psi at 28 days, maximum nominal size aggregate: 1 inch

Class B Concrete - f'c =

2,500 psi at 28 days, maximum nominal size aggregate: 1 inch

Fuller Mossbarge Scott & May

4,500 psi at 28 days, maximum Pre-Cast Concrete - f'c = nominal size aggregate: 1 inch

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- 56. Reinforcing steel shall have a minimum yield of 60,000 psi.
- 57. Concrete coverage for reinforcing, unless otherwise noted, shall be in accordance with ACI 318. latest edition.
- 58. Provide details in accordance with ACI 315. latest edition.
- 59. Dimensions for bar spacing are center to center of bar unless otherwise shown. Clearances are to the outside edge of the bar.
- 60. Class A Concrete shall be used for all reinforced concrete structures unless otherwise noted.
- 61. All concrete surfaces shall be finished to a smooth, sound surface. Surface defects due to forming shall be corrected.
- 62. All exposed concrete edges shall be chamfered 3/4 inches unless otherwise noted.
- 63. The storm water collection pipes between catch basins, manholes, and wet wells shall consist of solid wall high density polyethelene (HDPE) piping meeting the requirements of the specifications.
- 64. All storm water collection system manholes shall be four (4) feet inner diameter precast manholes as shown on the drawings.
- 65. Construction joints shall not be used at locations other than those shown on the plans unless prior approval is obtained from the OWNER'S REPRESENTATIVE.
- 66. Provide dowels, unless otherwise noted, in walls, slabs, footings, etc. for any concrete not placed at the time the original work is placed. Dowels shall be the same size as main reinforcement in concrete work and shall lap as noted:

<u>Minimum</u>	Splice Length
<u>Bar Size</u>	Grade 60 Stee
No. 3	1'-6"
No. 4	2'-1"
No. 5	2'-7"
No. 6	<i>3'-1"</i>
No. 7	4'-6"
No. 8	5'-2"
No. 9	5'-9"
No. 10	6'-5"
No. 10	7' 1"

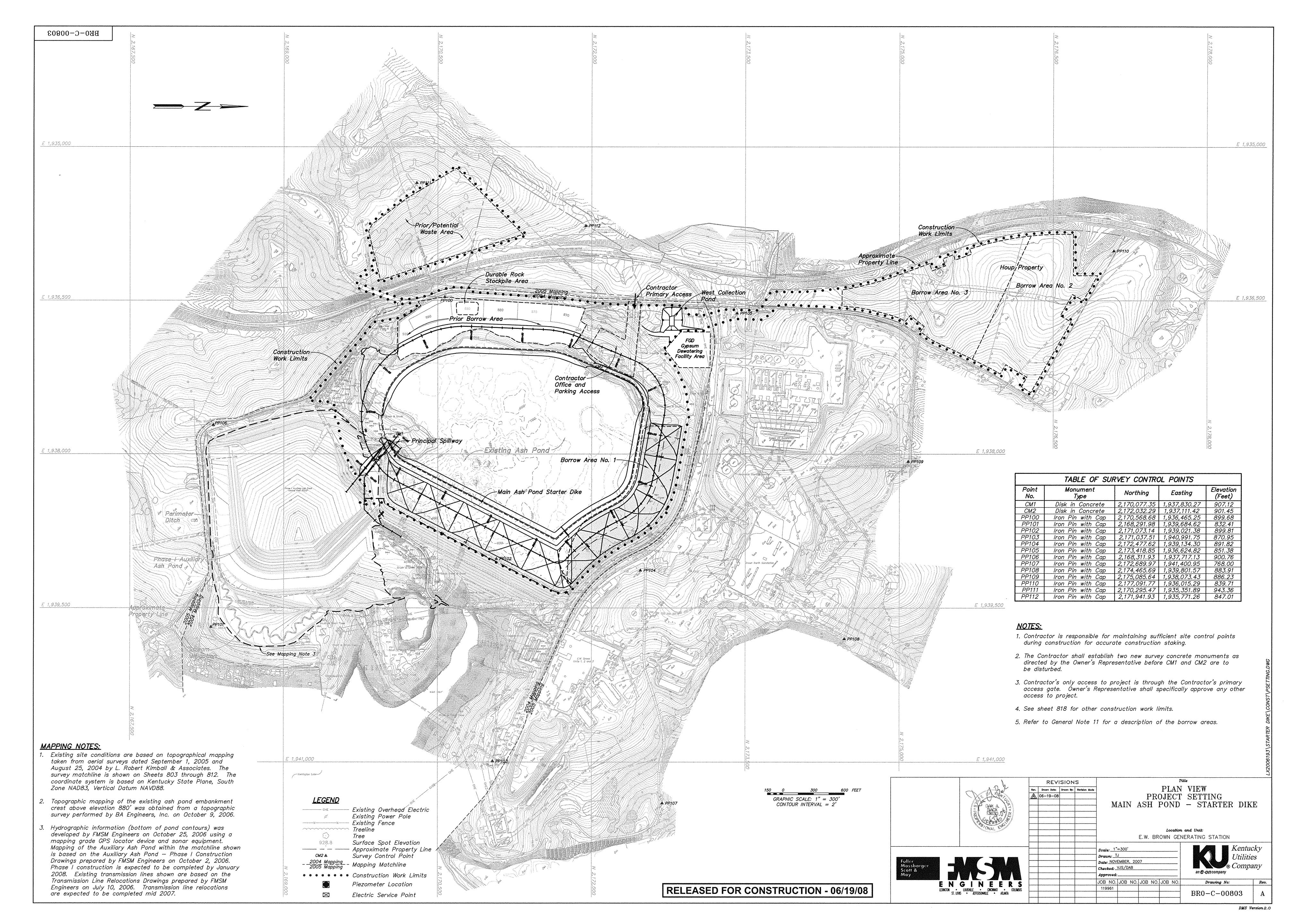
- 67. All dowels, anchor bolts, manhole steps, wall thimbles, and manhole frames shall be cast—in—place.
- 68. Rock anchor bolts shall be installed as quickly as practical after rock excavation of the riser foundation is complete. Initial installation of rock anchor bolts shall be completed under the direct supervision of a auglified manufacturer's representative and in the presence of the OWNER'S REPRESENTATIVE.
- 69. Rock anchor bolt prestressing shall not be conducted until the gel time of the resin for the ambient temperature has been surpassed.
- 70. Several above grade byproduct pipelines have been (or will be) installed along the crest and downstream toe/bench of the existing Main Ash Pond embankment during the Auxiliary Ash Pond construction project. All existing byproduct pipelines shall remain in place and shall not be disturbed. The pipeline alignments shown on the drawings are conceptual only, and the CONTRACTOR shall visit the site to observe the actual byproduct pipeline installations prior to bidding.
- 71. Existing topography shown on the drawings has been changed in some areas during the construction of the Auxiliary Ash Pond. Changes in topography include, but are not limited to, material stockpile areas, haul roads, and earth and rock borrow areas. The CONTRACTOR shall visit the site and observe the actual field conditions prior to bidding.
- 72. A field study was performed in 2005 to characterize the engineering properties of the ash deposits and to demonstrate the viability of construction access onto the saturated ash deposits. The CONTRACTOR shall refer to "Geotechnical Character ization of Ash Basin Deposits" Report prepared by FMSM Engineers, Inc. on September 18, 2006 for additional geotechnical data obtained from the field study.
- 73. Bottom ash stockpile stormwater runoff will not be allowed to flow offsite except to the existing ash ponds or through permitted discharge points. Some additional effort to route this runoff should be expected.
- 74. The west side sump will be constructed to contain stormwater runoff from the gypsum dewatering facility. The work also includes drop box inlet, manhole, and piping installation. The piping will connect to an OWNER installed wet well pumping system located outside the sump area. The CONTRACTOR will be responsible for all sump excavation, including the wet well structure. The CONTRACTOR will coordinate his work with the wet well installer. The CONTRACTOR will be responsible for back filling the wet well according to the specifications.
- 75. Trees displaced by the west side sump construction shall be replaced with white pine trees according to the drawings and specifications.
- 76. The LLDPE FML shall be attached to the polyethylene embed channels which are embedded into the pier and riser concrete structures as indicated on the drawings and described in the specifications. Clay and ballast layers shall be constructed at the riser also as noted in the drawings and specifications.
- 77. A temporary boom skimmer shall be installed around the riser according to the drawings and specifications. The temporary skimmer shall remain in use for the duration of this contract. The permanent skimmer noted in the specifications will be supplied and suspended on the riser during this project. It will be lowered into place during a future phase expansion at which time the temporary skimmer can be removed.

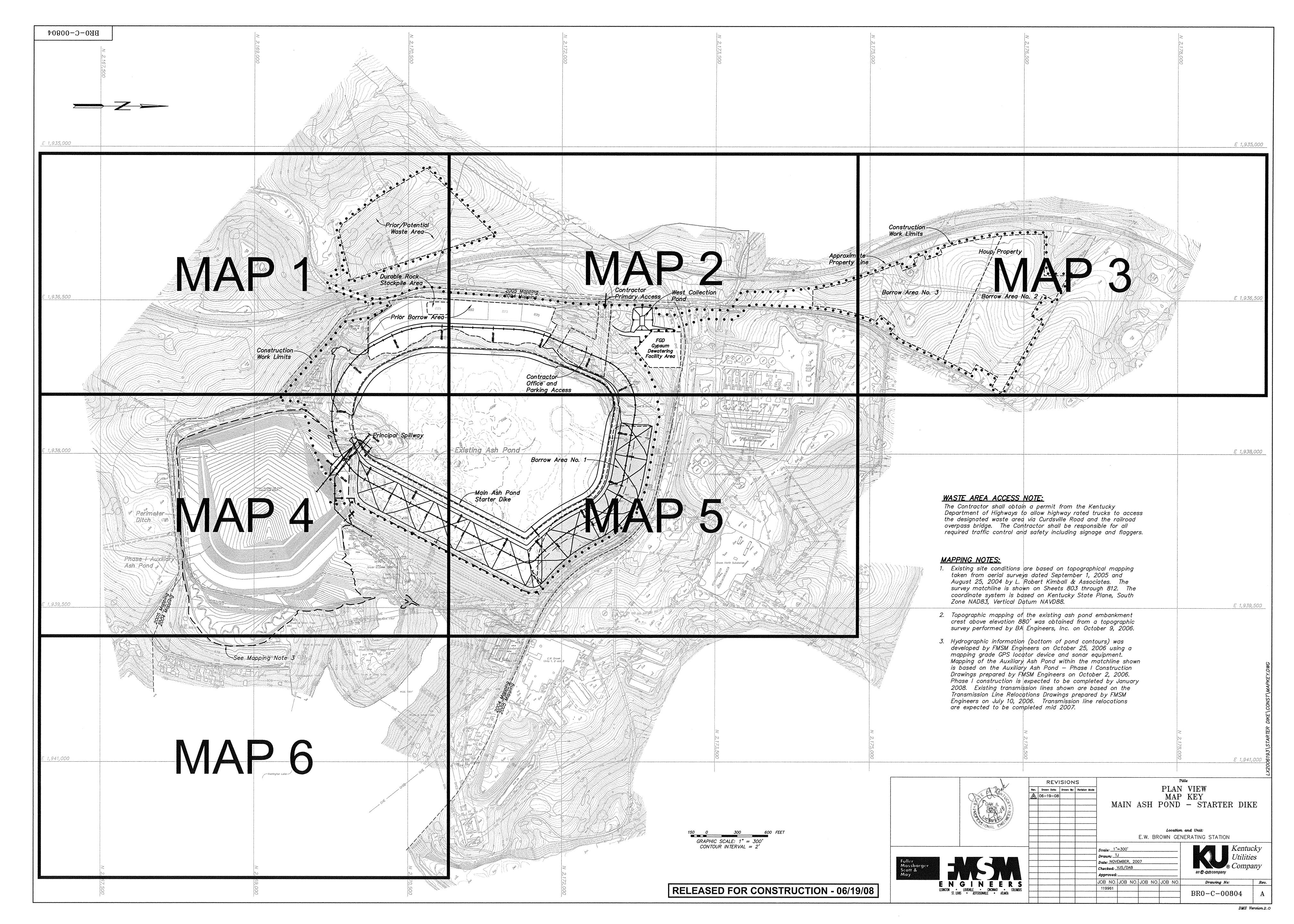
**REVISIONS** GENERAL NOTES Rev. Drawn Date: Drawn By: Revision Mode A 06-19-08 MAIN ASH POND - STARTER DIKE Location and Unit: E.W. BROWN GENERATING STATION

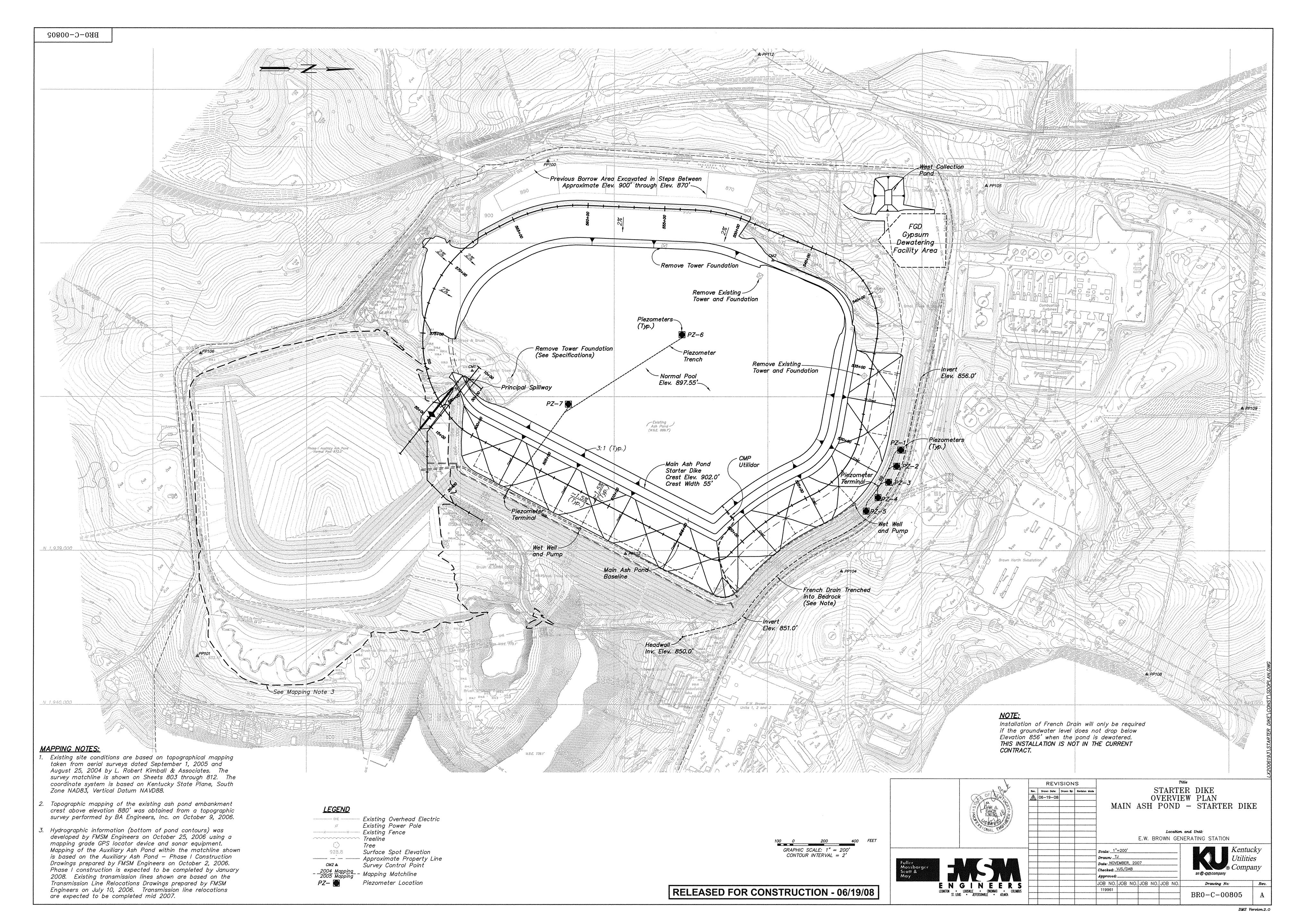
Date: NOVEMBER, 2007 Checked: VJS/DAB Approved: \_\_ JOB NO. JOB NO. JOB NO. JOB NO.

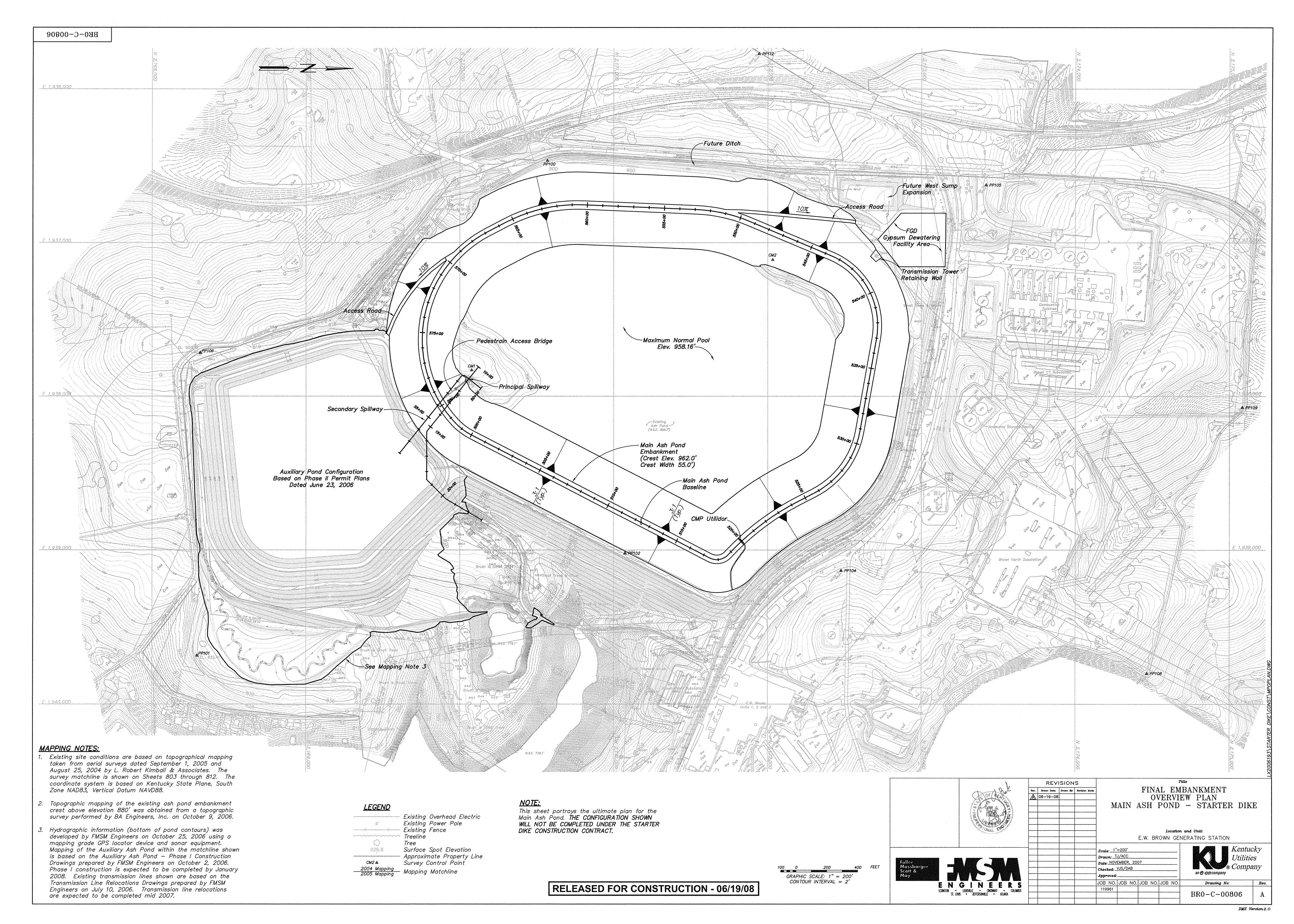
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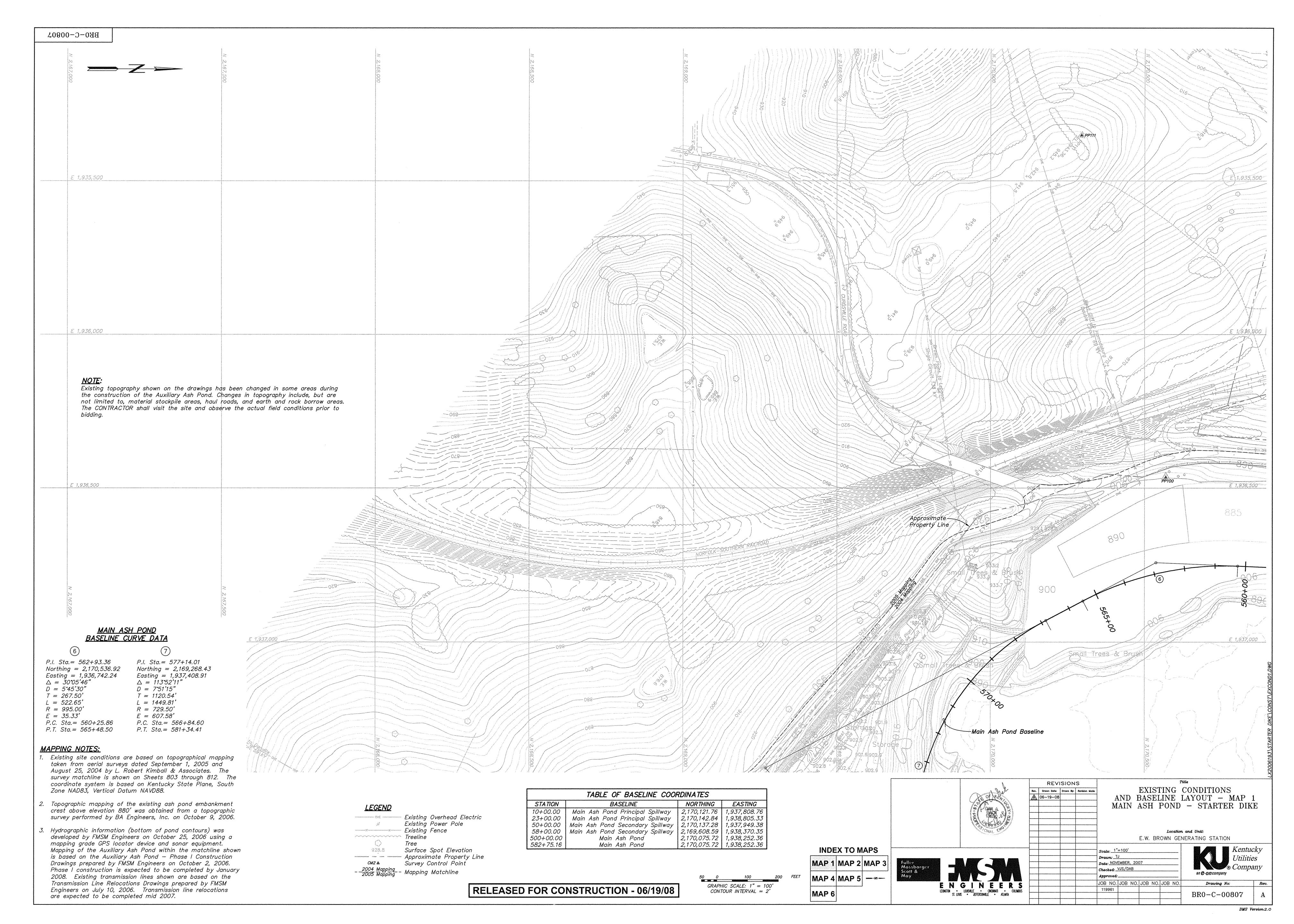
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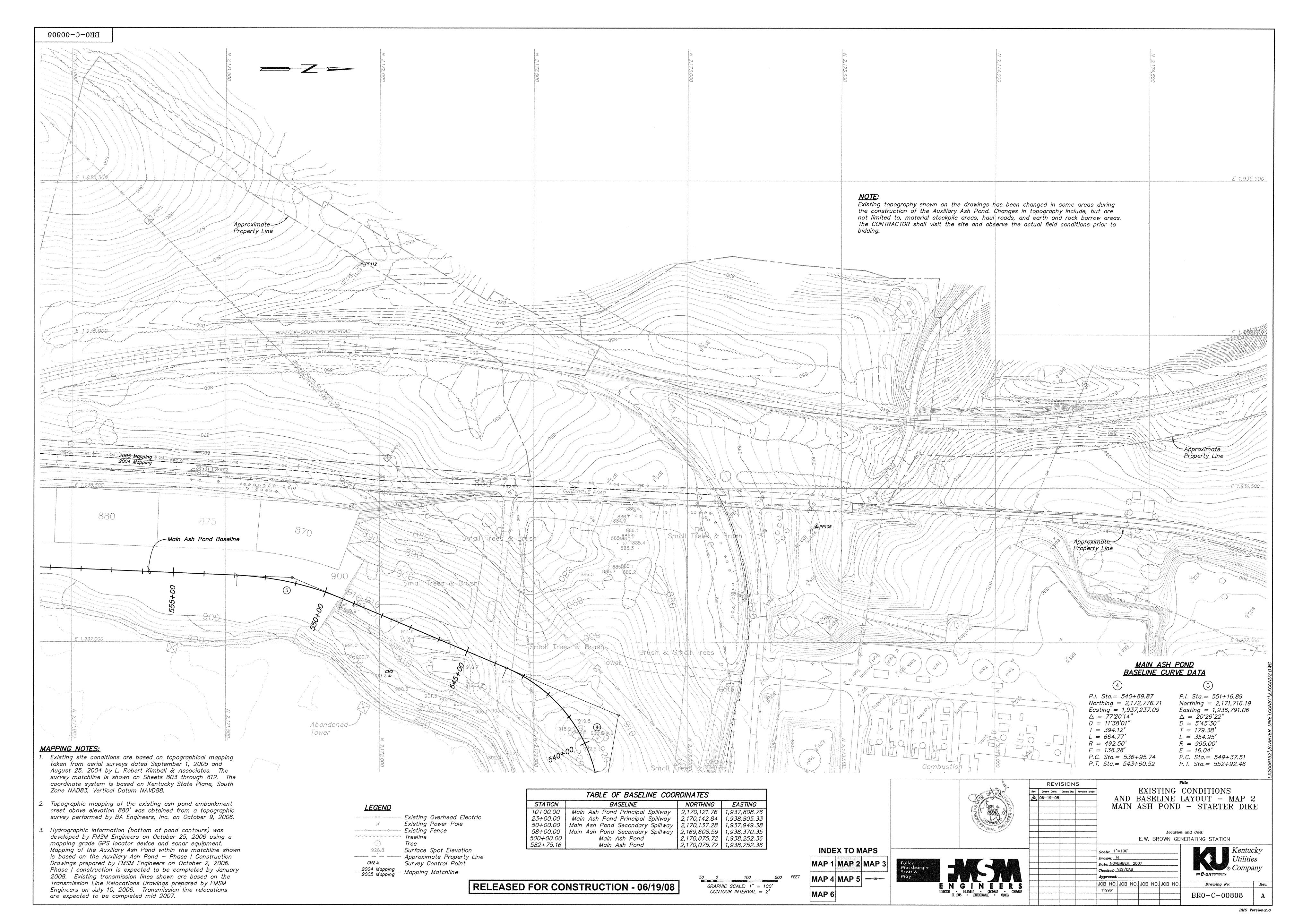


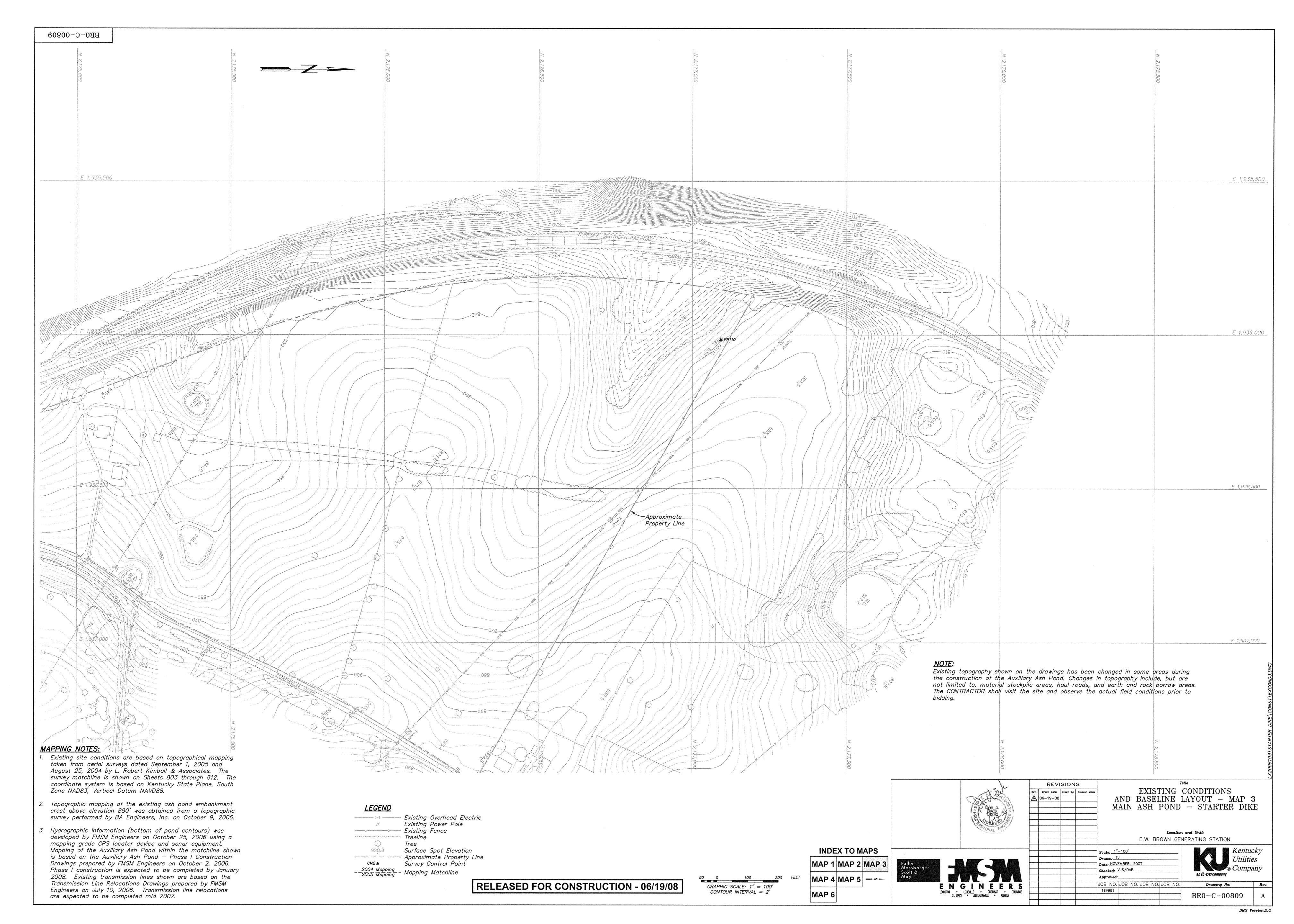


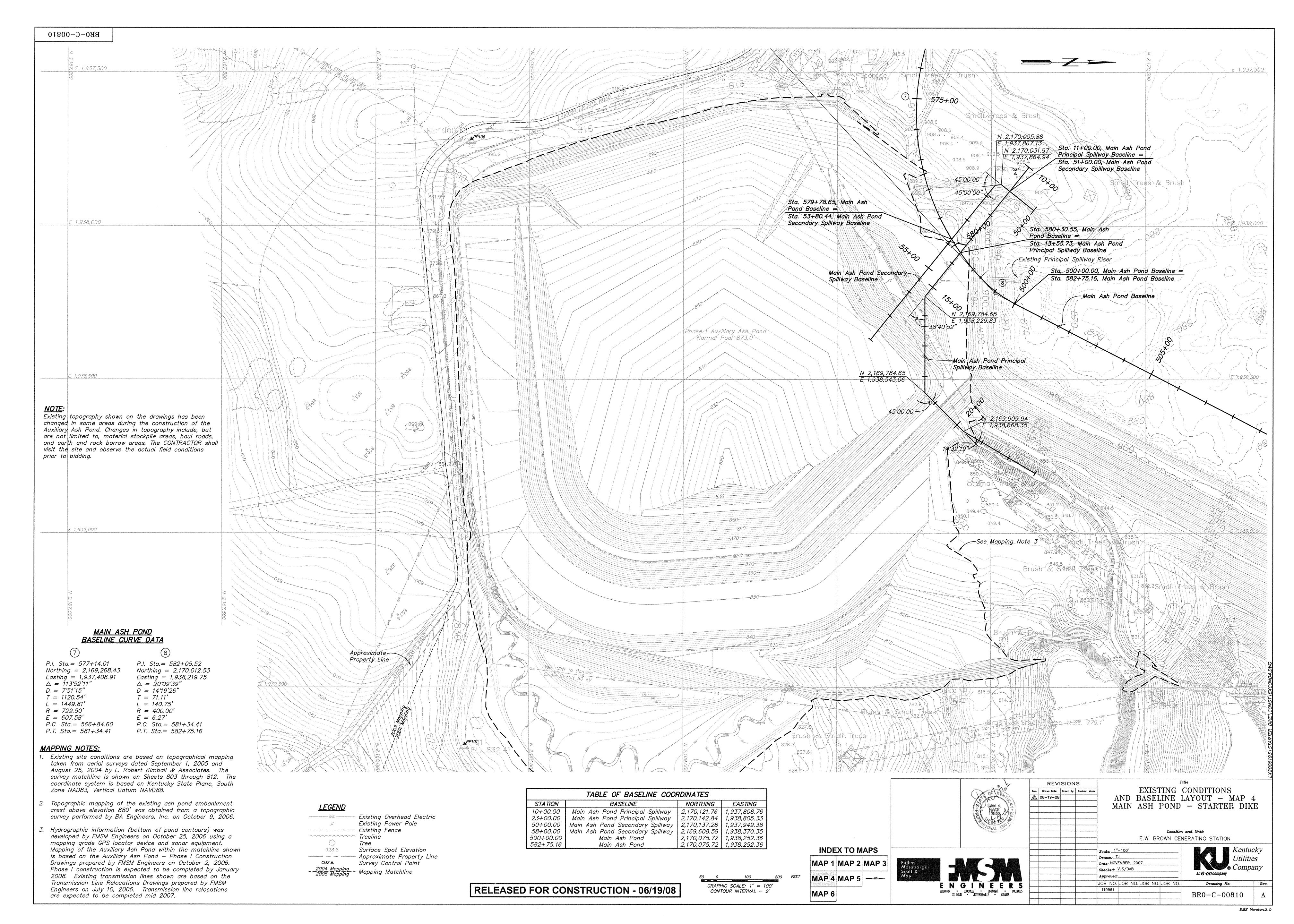


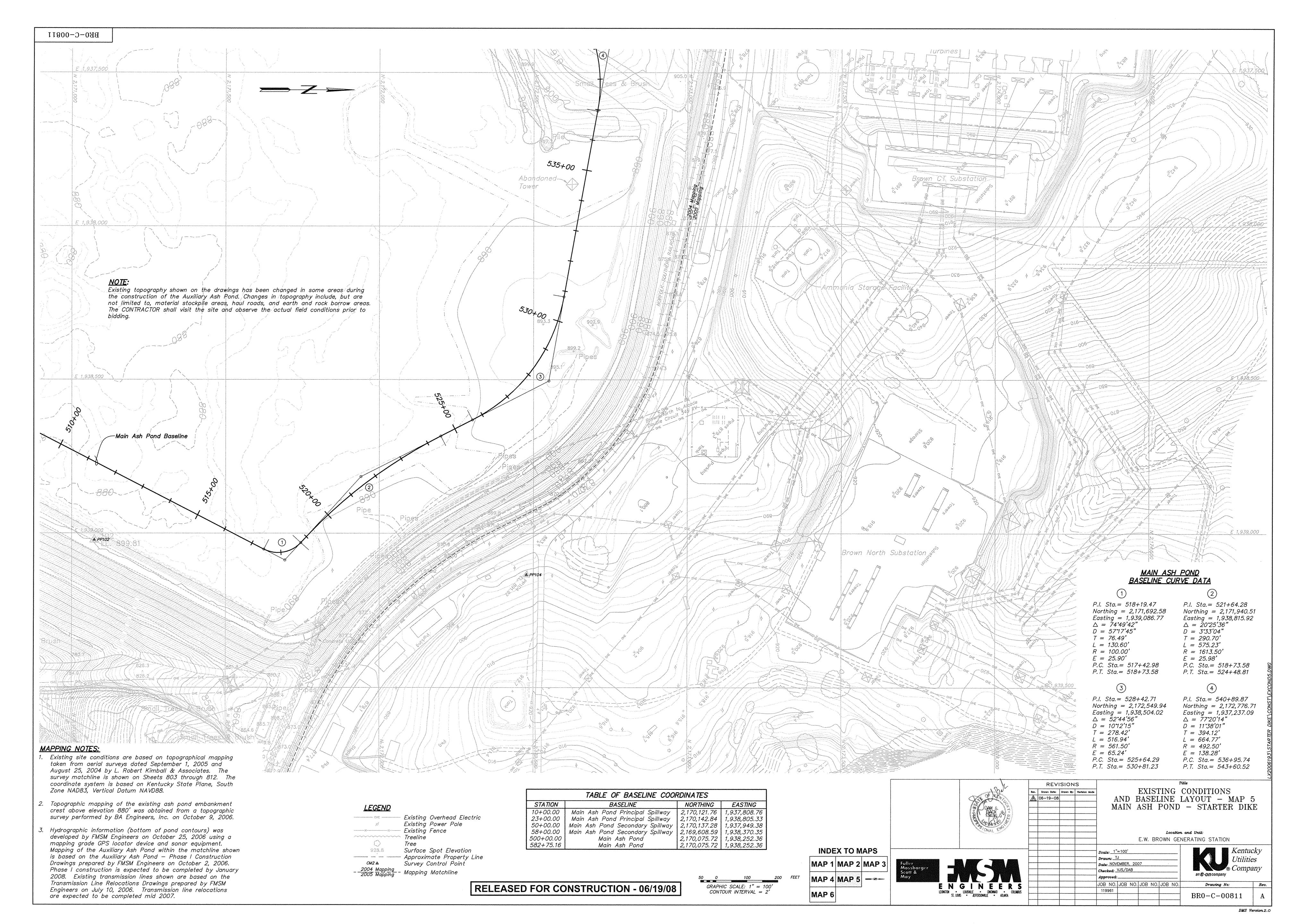


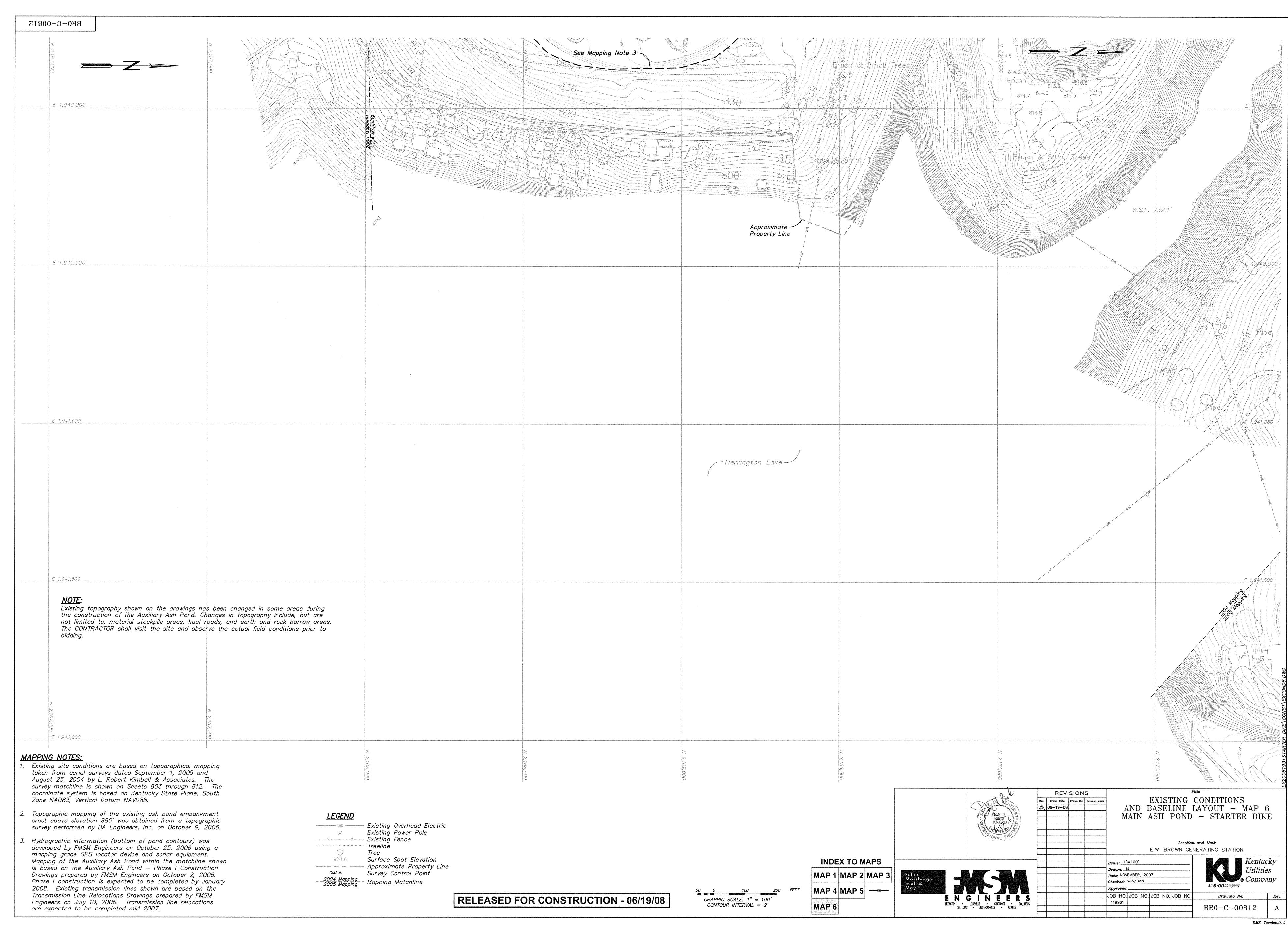


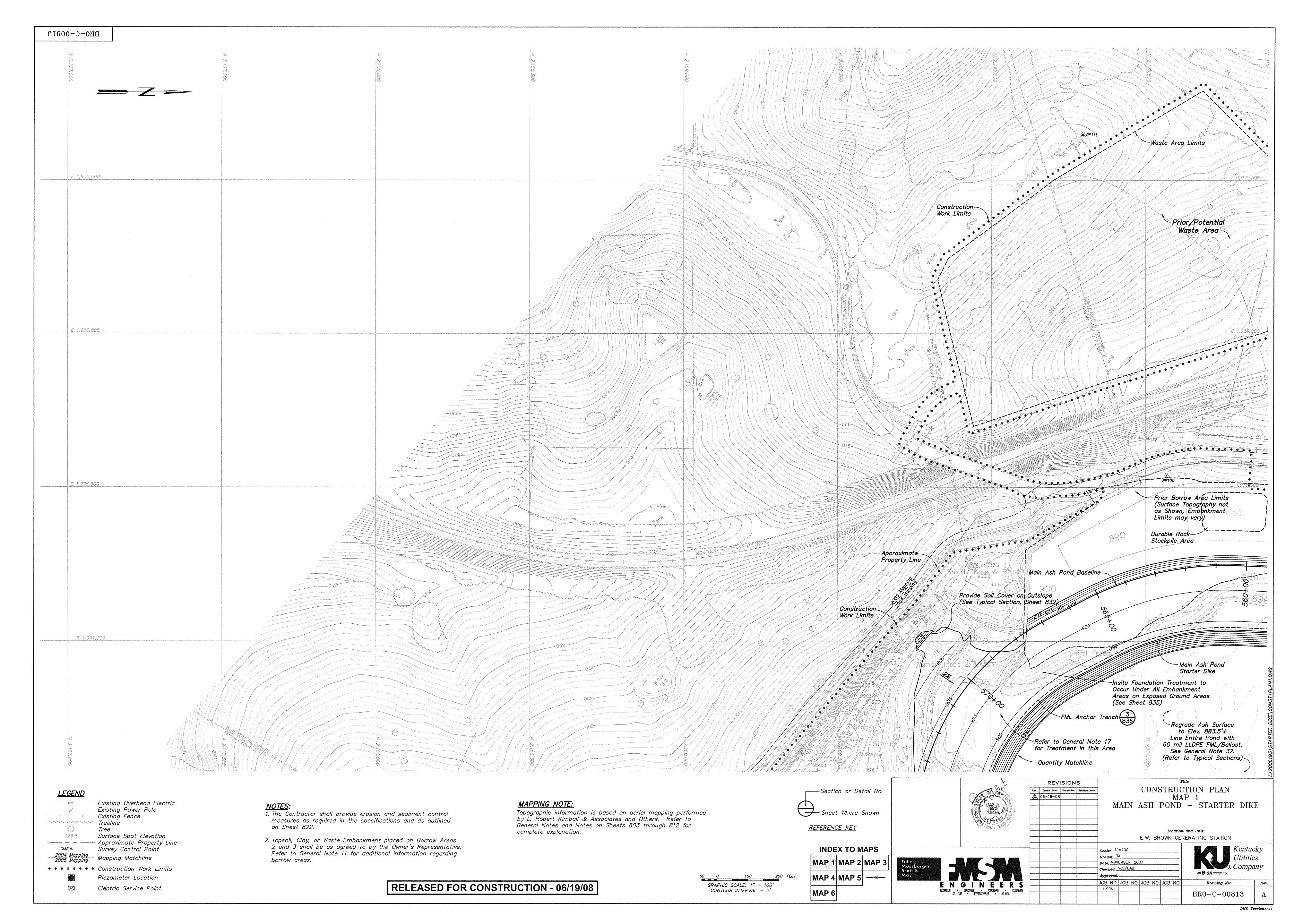


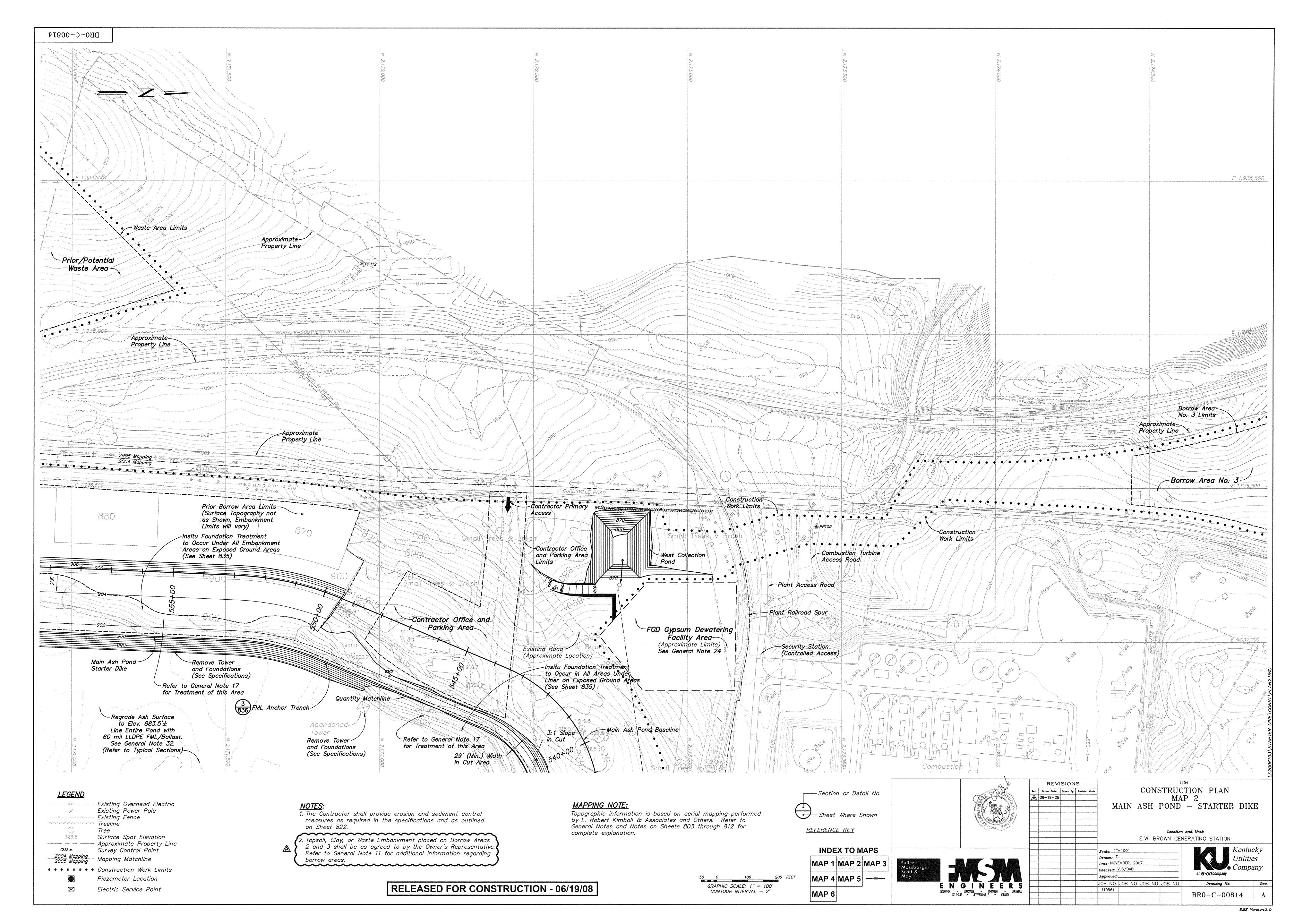


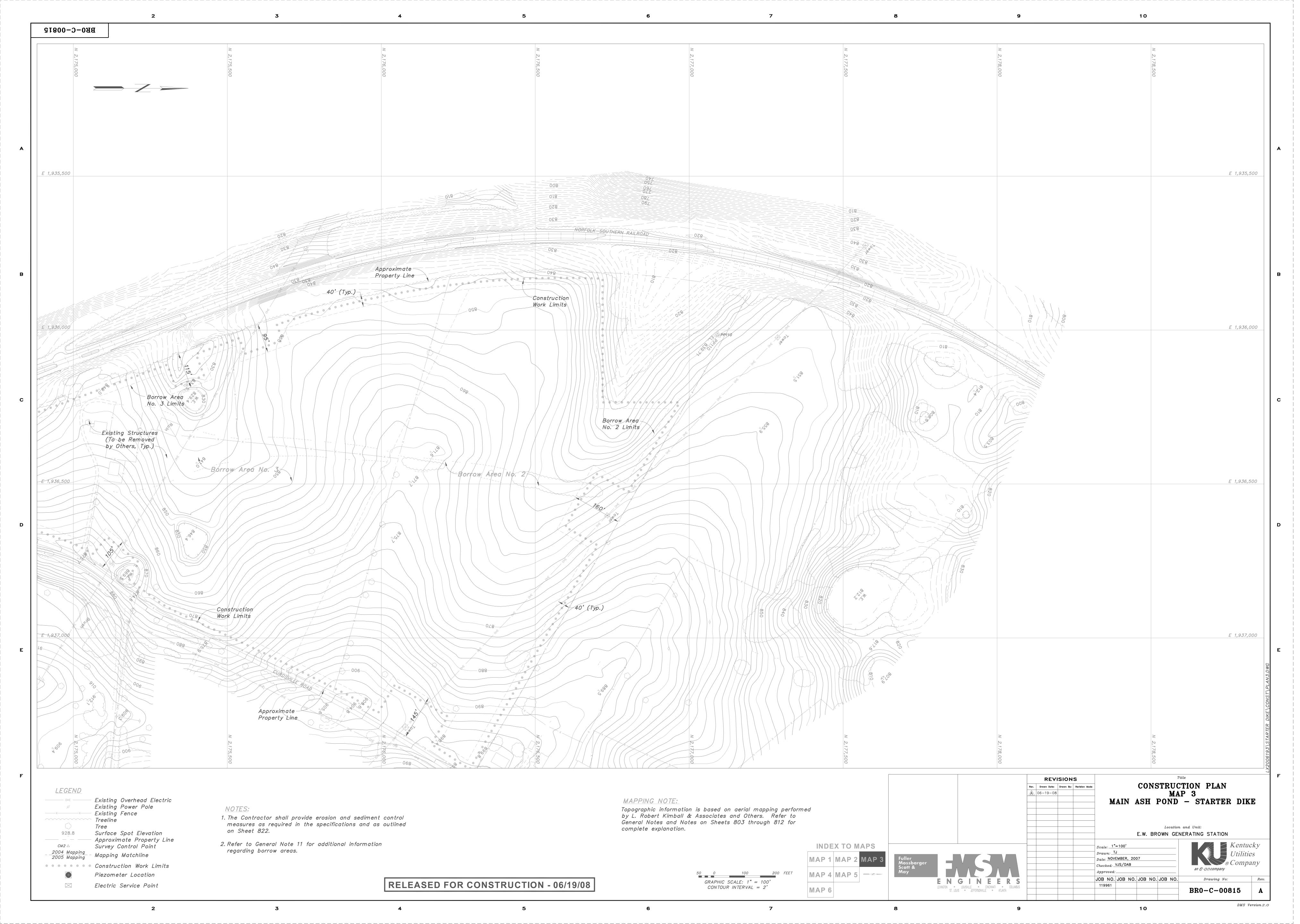


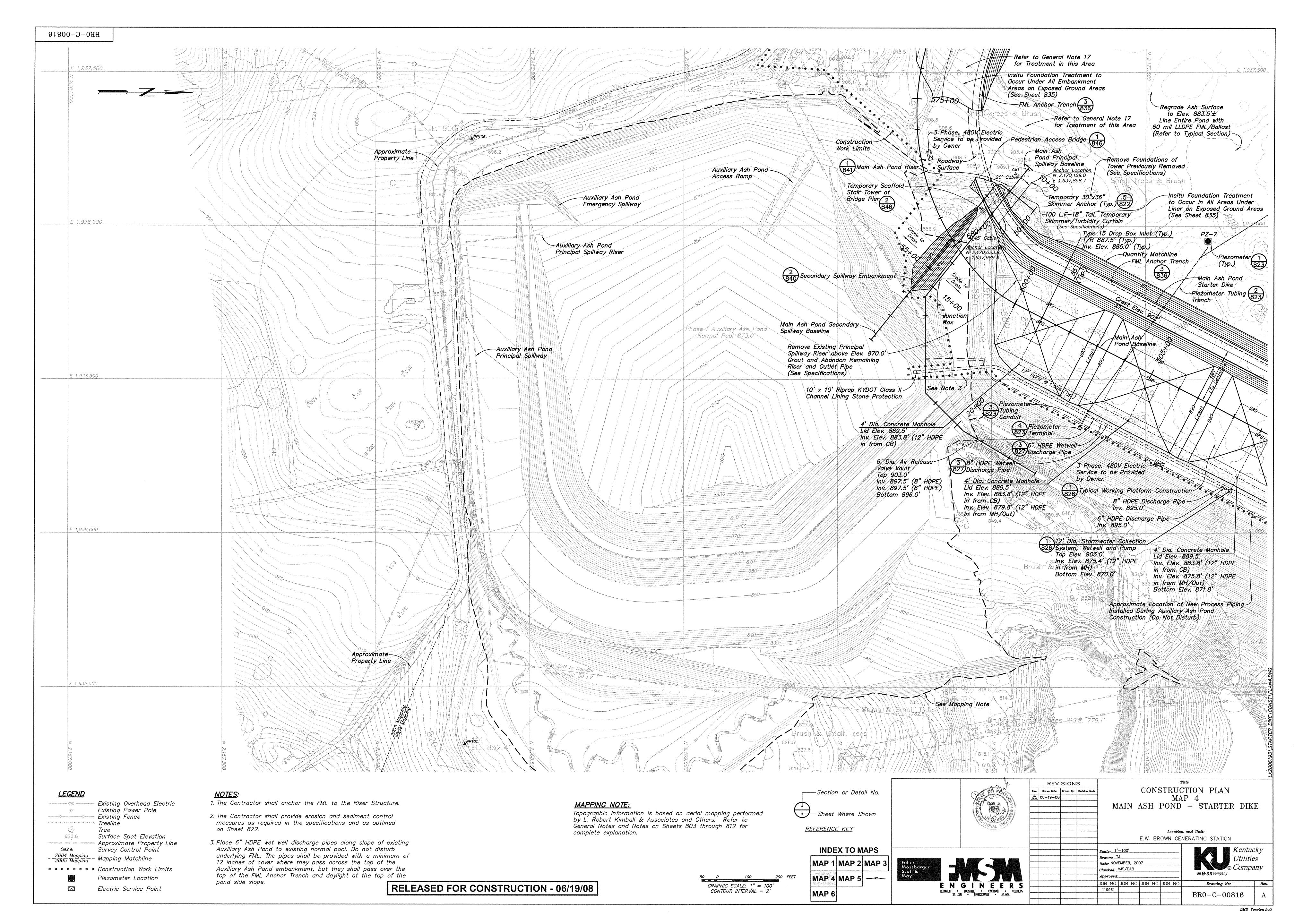


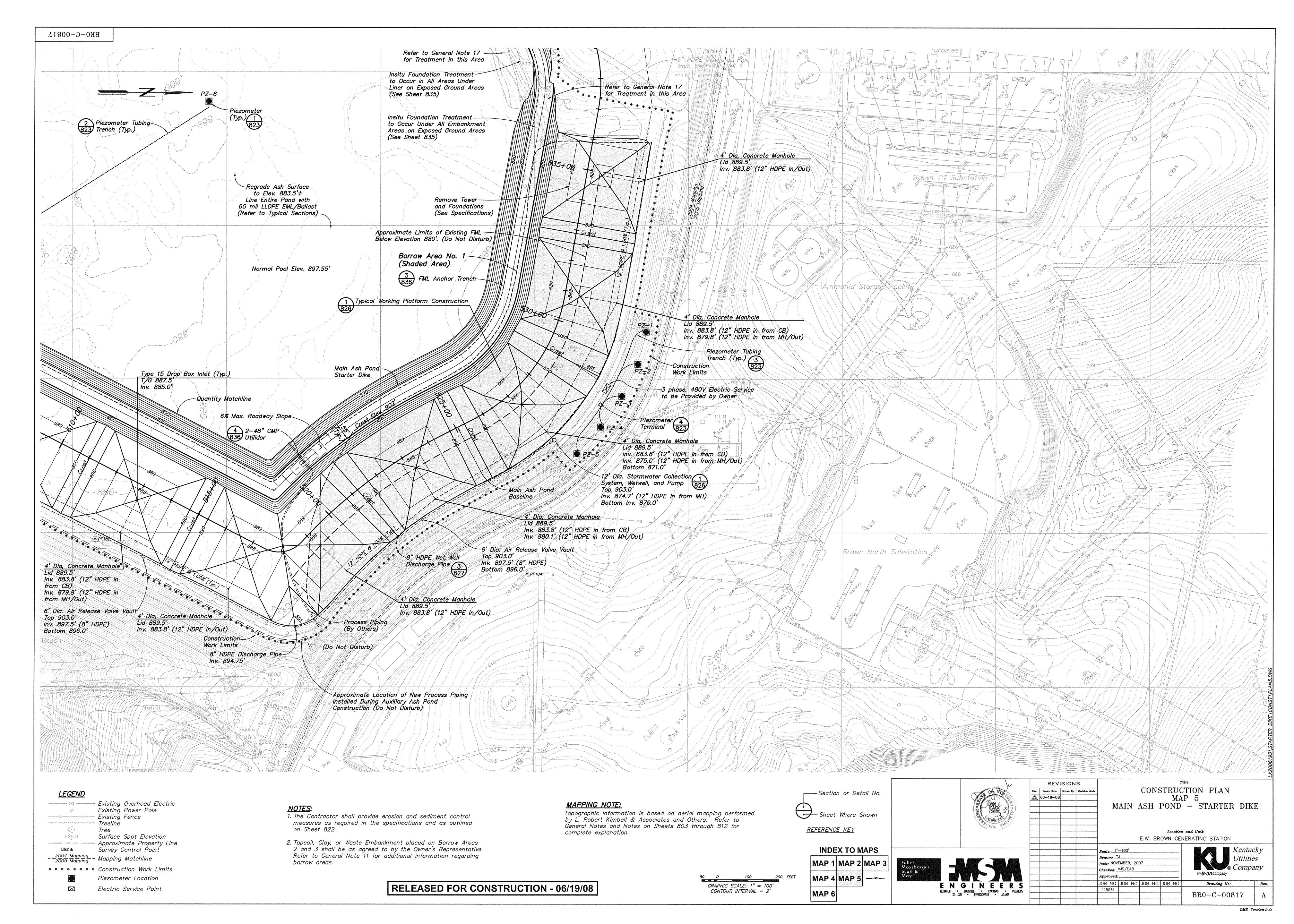


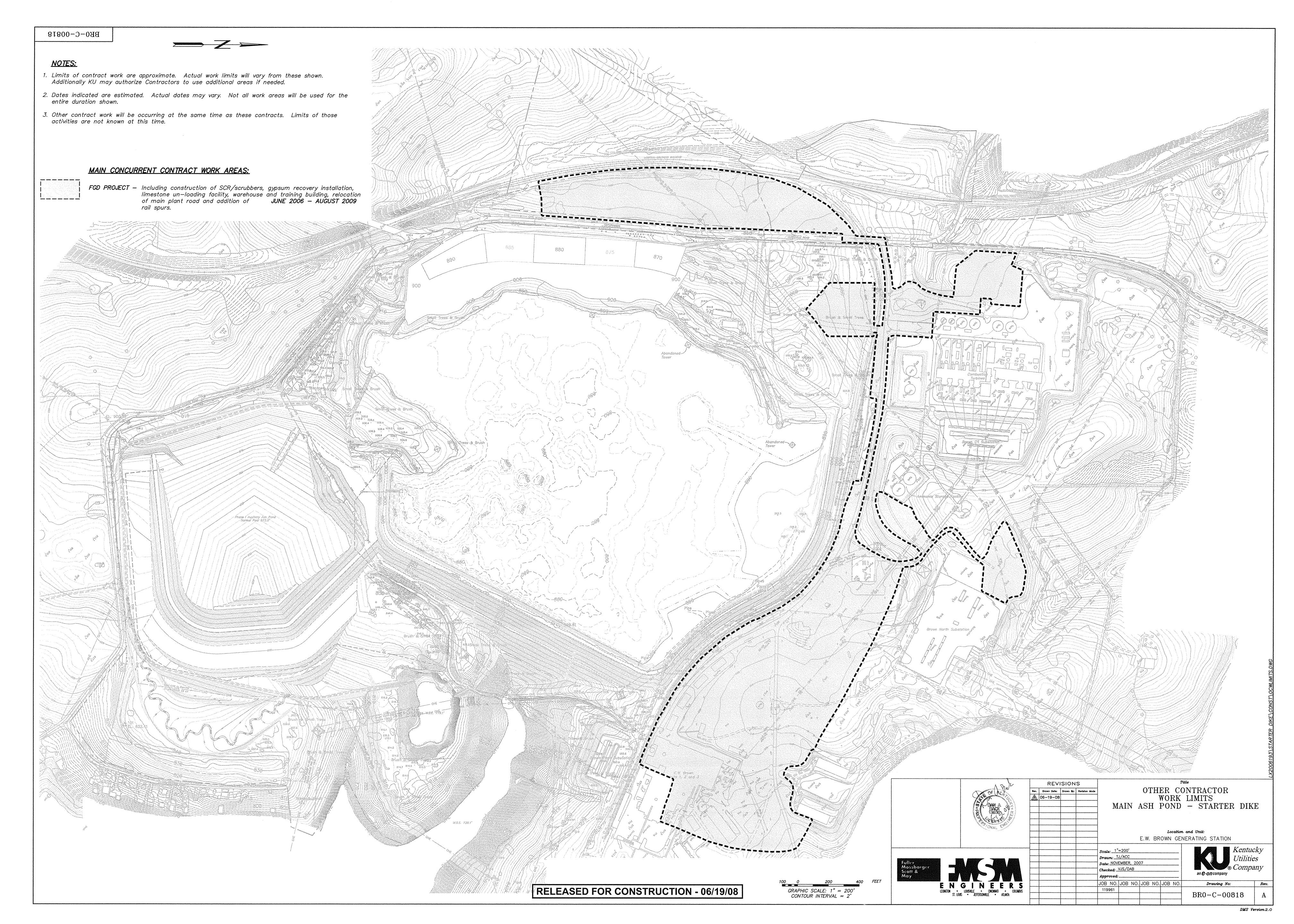






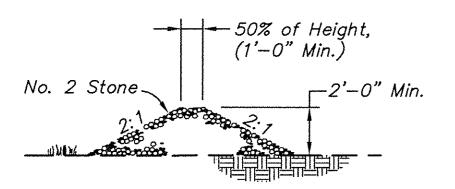






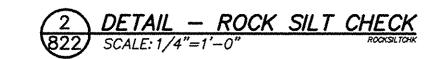
#### PROJECT SEDIMENT AND EROSION CONTROL NOTES

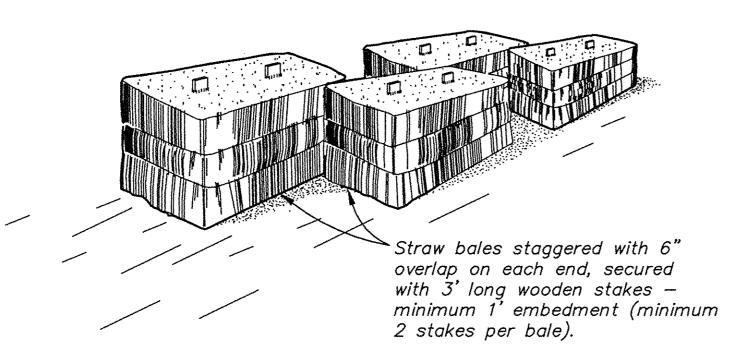
- Contractor shall adhere to the best management practices in "Kentucky Erosion Prevention and Sediment Control Field Guide" published by the Kentucky Transportation Cabinet.
- 2. Erosion and sediment control measures shown on these plans are conceptual only and shall be considered as the minimum. Prior to mobilizing to the site the Contractor shall submit, for approval, an erosion and sediment control plan specific to the job and his planned sequence of operation. This plan shall consist of an appropriate, (to scale) plan view, details, specifications/materials and written text describing the installation and maintenance procedures. This plan shall be resubmitted with any necessary modifications when construction sequencing is modified for any reason.
- 3. All disturbed areas, including borrow areas, existing roadways, new roadways and plan excavation/embankment areas, shall be protected by adequate sediment control elements. Sediment control elements for an area shall be installed and accepted prior to initial disturbance in that area.
- 4. Contractor shall provide maintenance for all Erosion Control Devices. After each rain, and at least weekly, Contractor shall inspect and make required repairs. Silt fencing shall be cleaned when silt buildup reaches 1/3 of silt fence height.
- 5. Refer to project specifications for additional erosion and sediment control requirements.



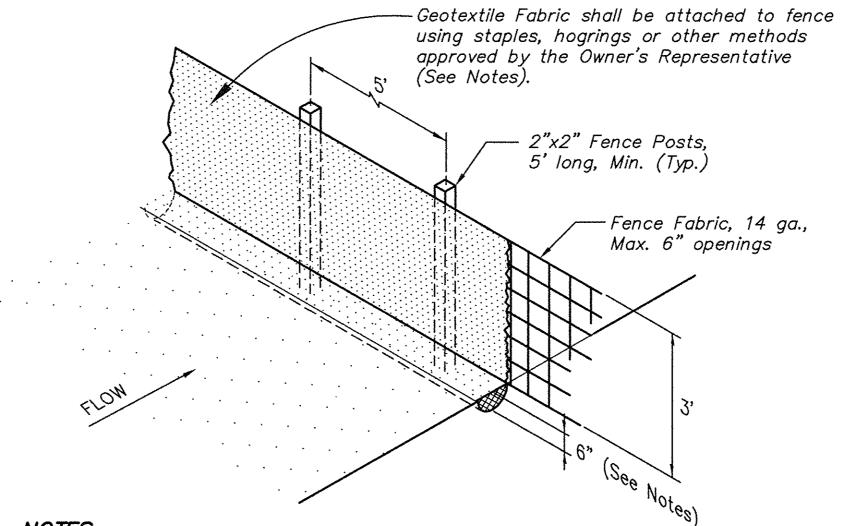
### NOTE:

Dimensions shown are minimums.
Larger rock silt checks will be
required where significant watershed
areas are to be protected. Rock Silt
Check Structures 5 feet or more
in height shall be provided with a
core of Class II Channel Lining.





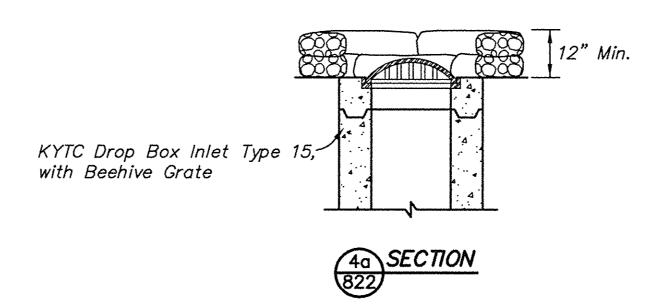




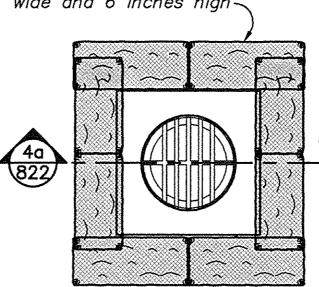
TES:

- 1. The bottom 12 inches of the fabric shall be buried in a 6-inch trench cut into the ground or covered by 6 inches of fill material to prevent sediment from escaping under the fence. All earthwork shall be on the upstream side of the fence.
- 2. Geotextile fabric shall meet the following specifications: Grab strength (ASTM D 1682) — 100 lbs. Min., Width — 4' Min., Bursting strength (ASTM D751) — 150 p.s.i. Min., Flow rate (KM64—106) — 0.3 gal./sq. ft., Retention efficiency (KM64—106) — 75%





KYTC No. 57 Crushed Stone contained in geotextile fabric bags (3 mesh), approximately 24 inches long, 12 inches wide and 6 inches high—

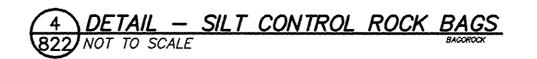


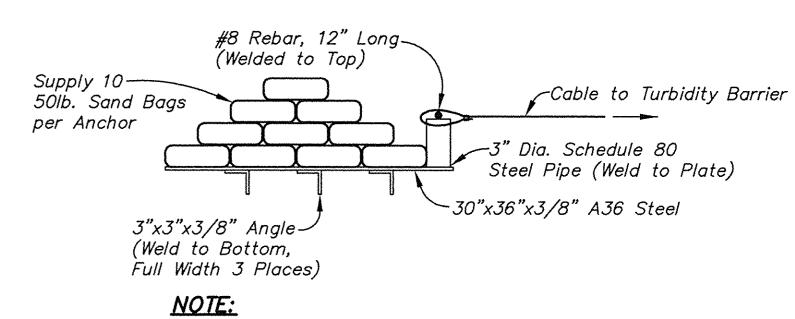
<u>PLAN</u>

#### NOTES:

- 1. Place bags such that there are NO gaps between bags.
  Bags may be single or double layers, as the situation dictates.
  2. Owner Representative shall approve/observe installation of
- silt control bags.

  3. Any damaged/non-approved silt control bags shall be replaced at the Contractor's expense.

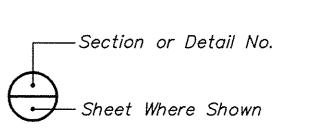




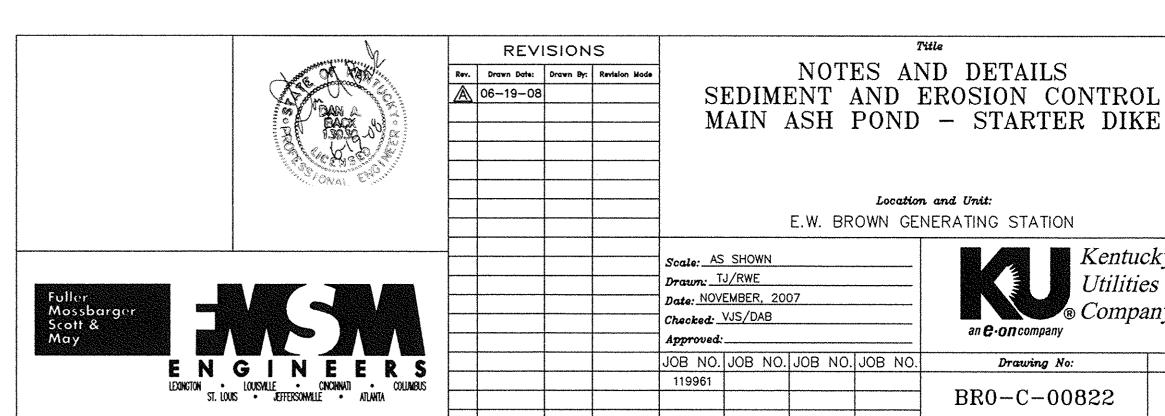
All steel parts to be hot dip galvanized.

5 ANCHOR DETAIL - TEMPORARY SKIMMER/TURBIDITY CURTAIN
822 SCALE: 1"=1'-0"

BAGANICHOR DETAIL - TEMPORARY SKIMMER/TURBIDITY CURTAIN
BAGANICHOR DETAIL

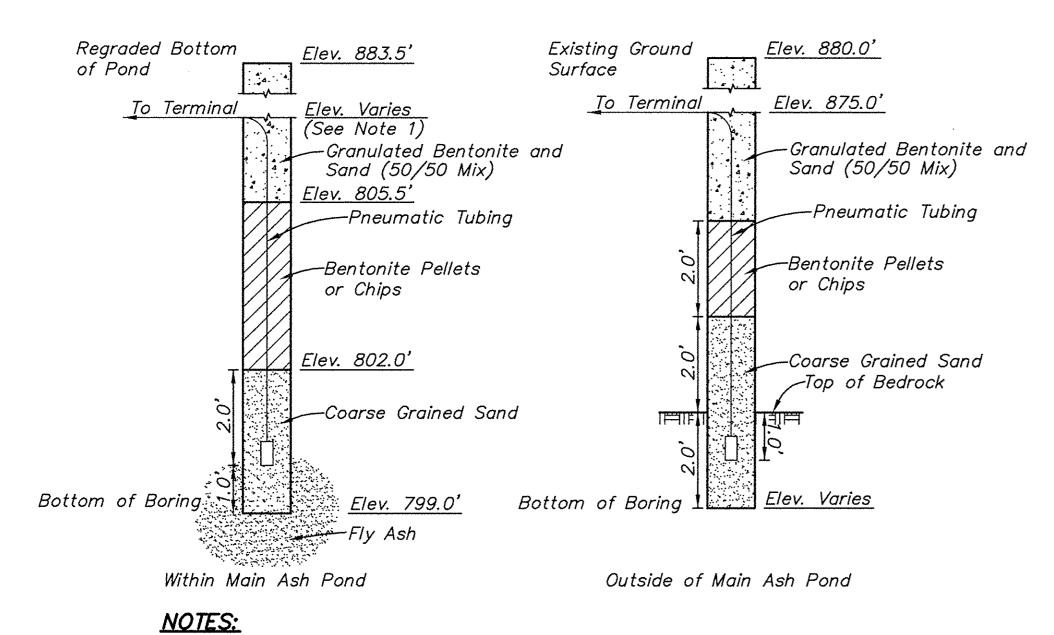


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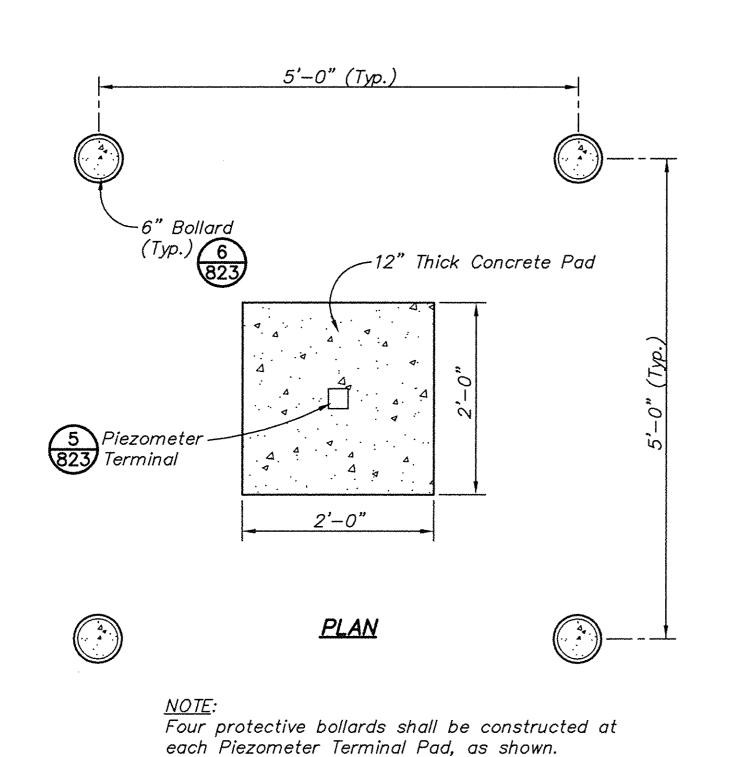
DMS Version 2.0



1. Minimum cover depth of 5 feet to be determined by Contractor to avoid any future construction and/or as directed by Owner's Representative.

2. Backfill of Each Piezometer shall be observed/approved by Owner/Engineer

1 DETAIL - PIEZOMETER INSTALLATION
823 SCALE: 1/2"=1'-0"



4 PLAN - CONCRETE PAD AND BOLLARD

823 LAYOUT AT PIEZOMETER TERMINAL

SCALE: 1" = 1'-0"

Zone 2 Zone −Max Rock size 1 1/2" 24" Min. Depth (Typ.)— ∠Zone 1 Twin Tubing bound in-Polyethylene Jacket (Min. of 1) (Typ.) . 12" (Min.) 12" (Min.) Tubing laid within Pond Tubing laid outside of Pond Zone 1 - Fly Ash Zone 2 - Earth 1. Backfill material shall be hand compacted to 95% Standard Proctor Value. 2. Installation and Backfill to be observed/ approved by Owner/Engineer. 3. Tubing shall be laid in trench allowing

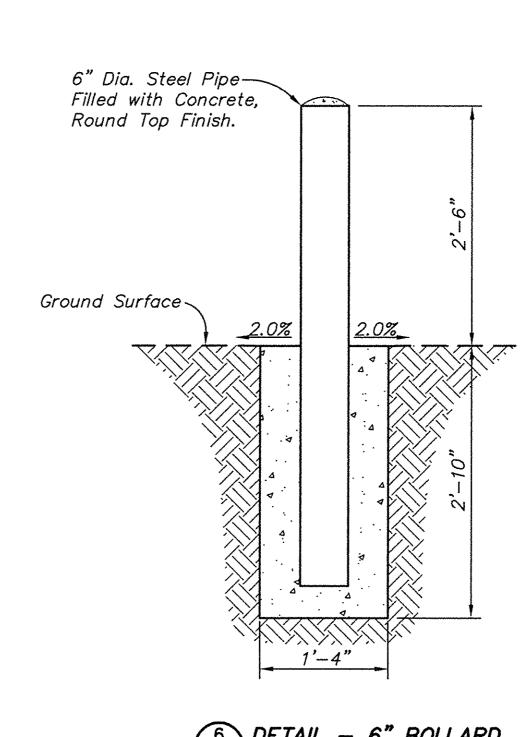
5% slack length (Typ.)

2 DETAIL — PIEZOMETER TUBING TRENCH 823 SCALE: 1"=1'-0"

Contractor to provide adequate means

to prevent damage to pipe during and

after installation.



PIEZOMETER LOCATION TABLE 2172863.2 1938346.4 2172836.7 1938451.4 2172784.7 1938555.5 2172716.1 1938655.9 2172638.8 1938743.0 2171441.6 1937595.9 2170702.0 1938048.1 854.0 *854.0* PZ-4 PZ-5 854.0 854.0 PZ-6 PZ-7 800.0 800.0

\_Roadway Surface

-Contractor to provide adequate means to

prevent floating of

cradle.

6" (Min.)

. 12" (Min.)

CONCRETE ENCASEMENT (TYP.)

Zone 1 - No. 9 Stone

1. Installation and Backfill to be observed/

2. PVC joints shall be threaded and sealed

3. Tubing shall be laid in pipe allowing 5%

slack length (Typ.) such that no kinks

3 DETAIL — PIEZOMETER TUBING CONDUIT TRENCH 823 SCALE: 1"=1'-0"

or pinch points are created or developed within the tubing or at the entrance and

such that water and/or concrete can not

approved by Owner/Engineer.

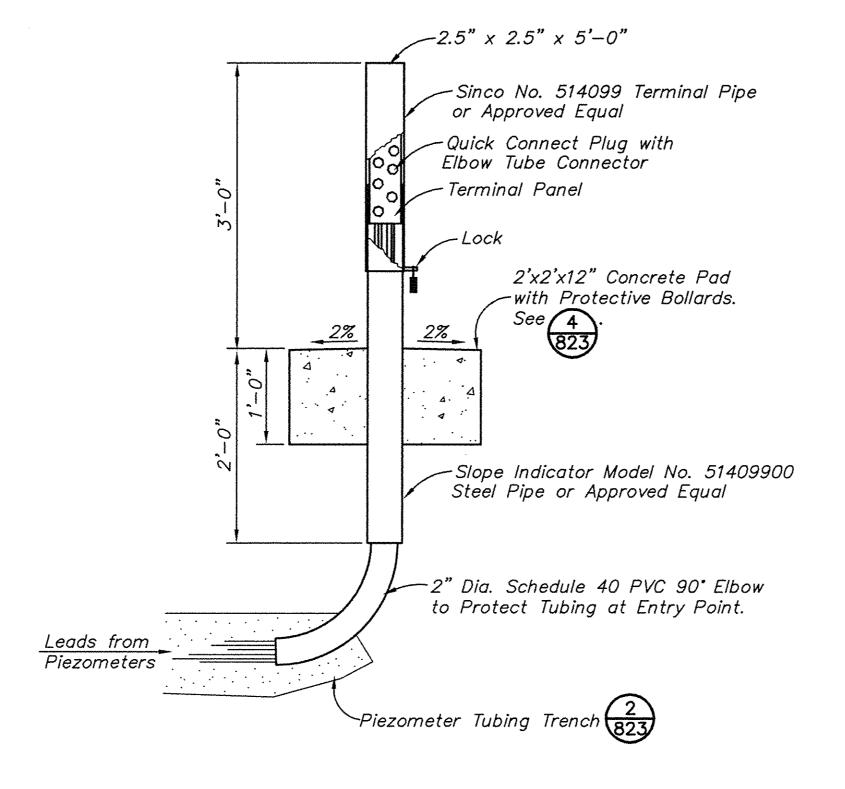
exit points of the conduit.

enter pipe.

4" O.D. Schedule 80 PVC—Piezometer Tubing—

pipe when installing

\* Tip elevations to be established at time of installation. Values listed are estimated minimum elevations which are likely to occur.

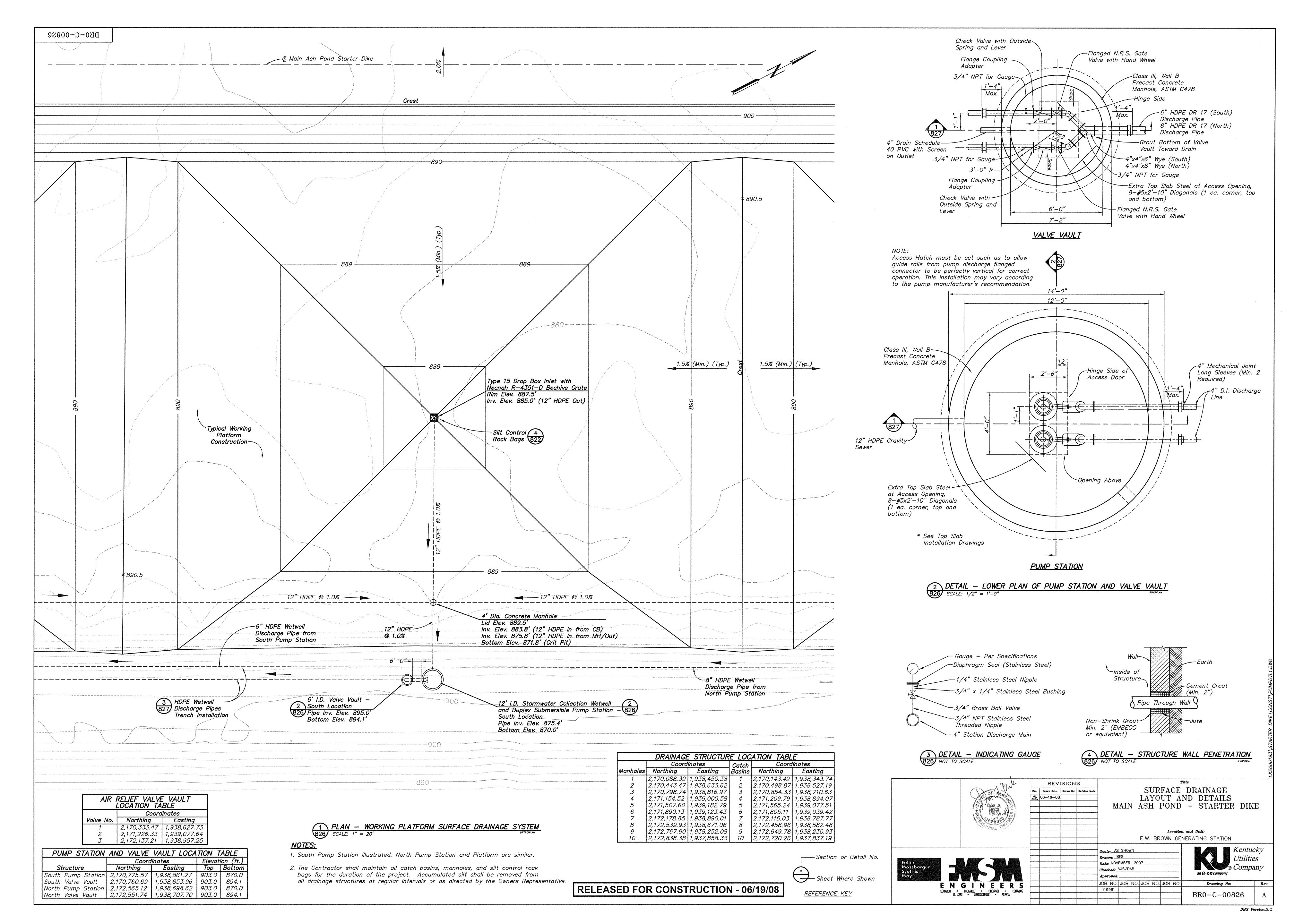


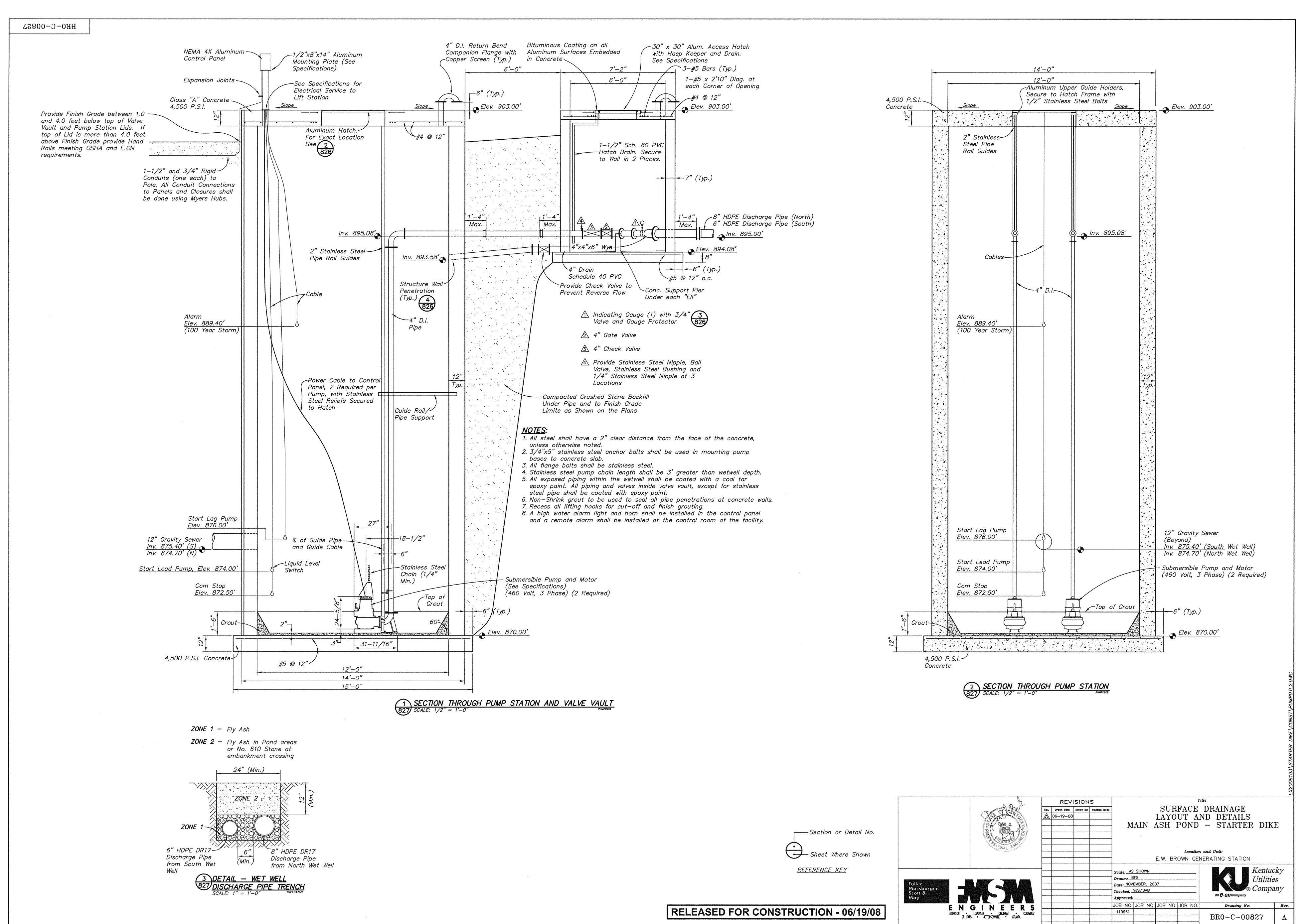
5 DETAIL — PIEZOMETER TERMINAL 823 SCALE: 1" = 1'-0" PZTERMINAL

6 DETAIL - 6" BOLLARD 823 SCALE: 1" = 1'-0" BOLLARDS

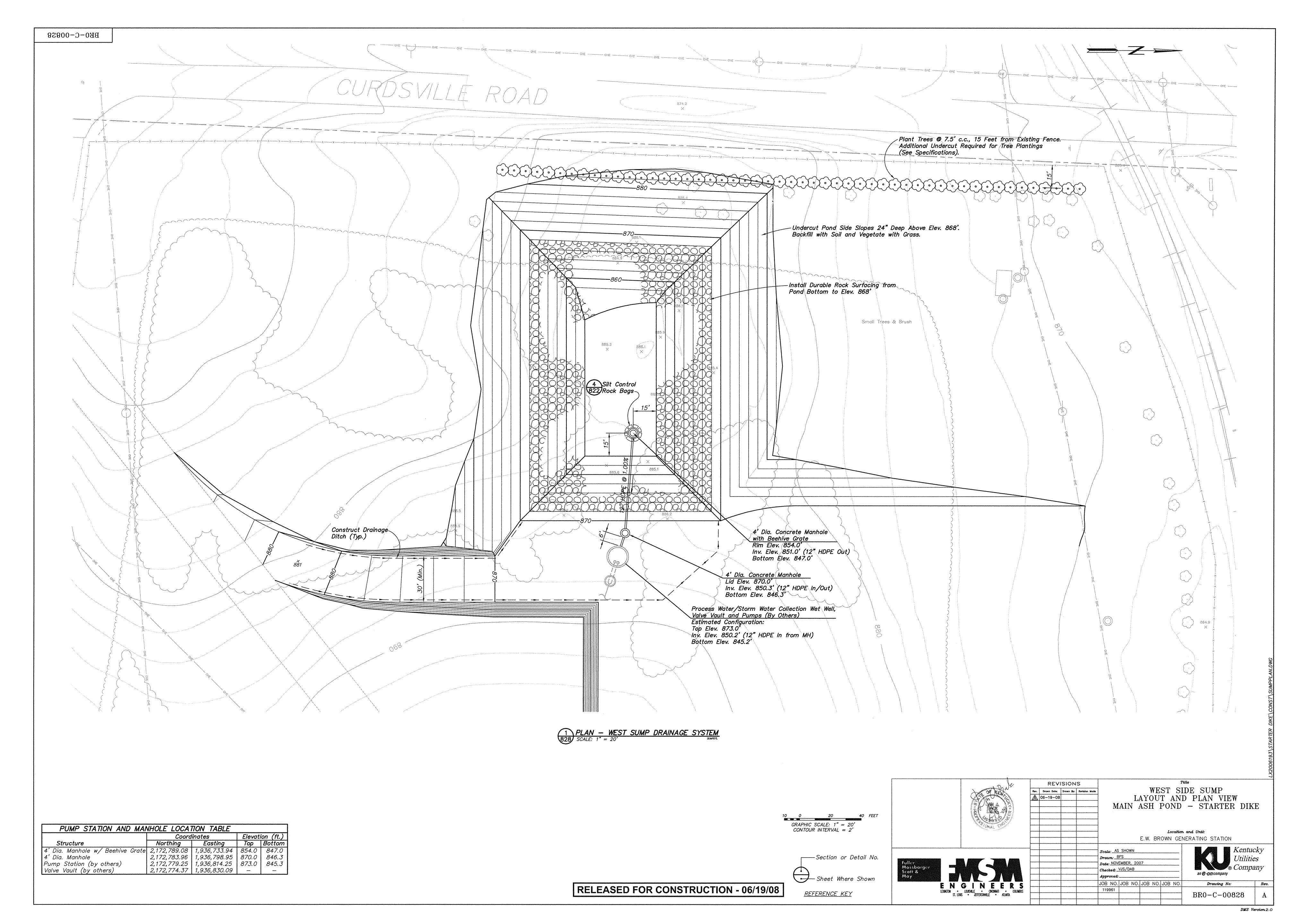
Section or Detail No.
Sheet Where Shown
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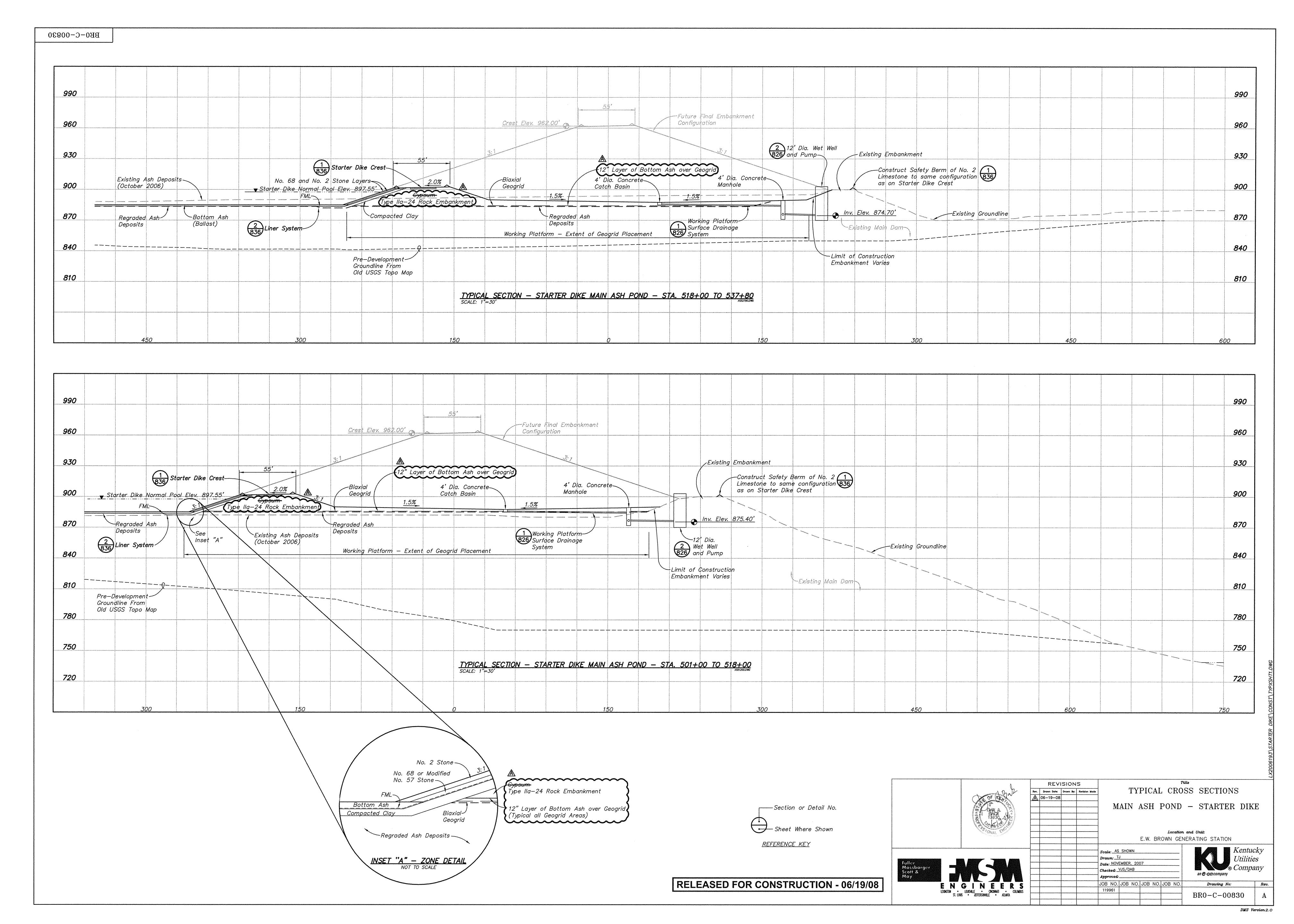
1			REV	ISION	S	PIEZOMETER DETAILS				
		Rev.	Drawn Date:		Revision Made					
			06-19-08			MAIN	ASH	POND	- STARTER DI	ΚE
							E.W. BI		n and Unit: NERATING STATION	
Fuller Mossbarger Scott & May					Scale: AS SHOWN  Drawn: RWE  Date: NOVEMBER, 2007			Kentuck Utilities ® Compar		
						Checked: GKA/VJS Approved:			an <b>e-on</b> company	
			1	1		JOB NO. JOB N	_		Drawing No:	

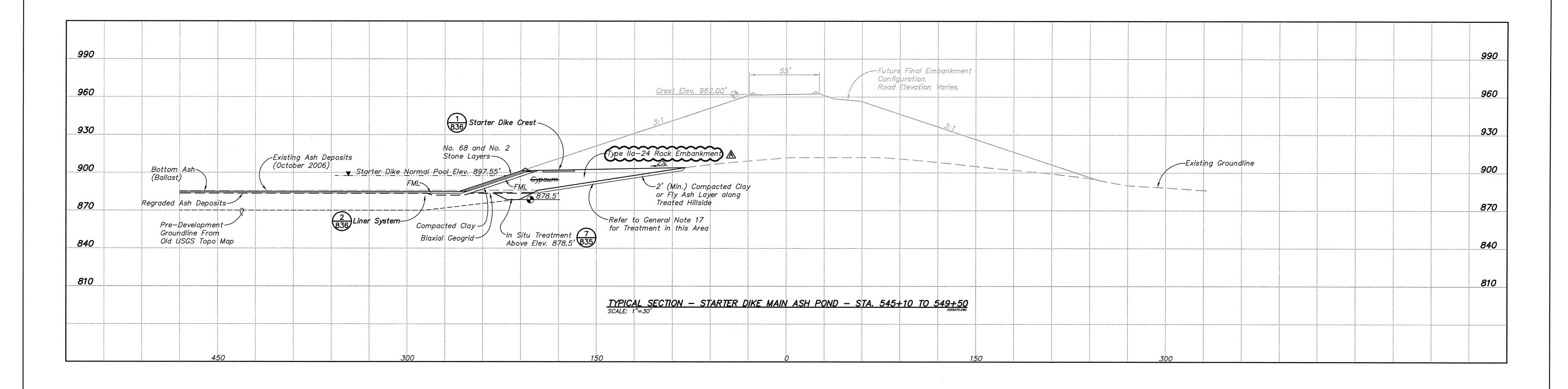


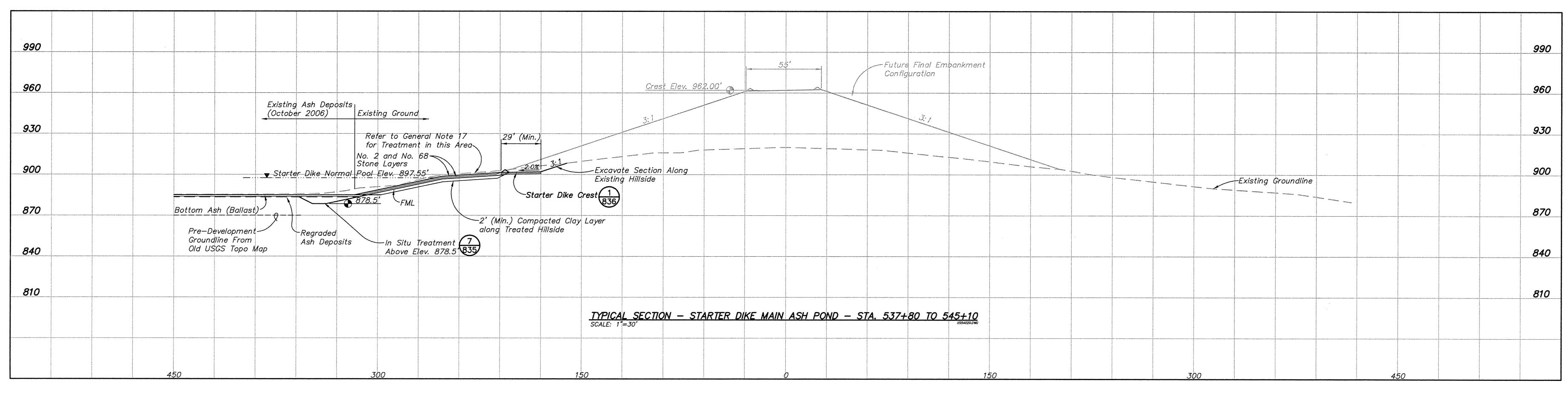


DMS Version 2.0









Section or Detail No.

Sheet Where Shown

REFERENCE KEY

REVISIONS

THE

TYPICAL CROSS SECTIONS

MAIN ASH POND — STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN

Drawn: JJ

Date: NOVEMBER, 2007

Chacked: VIS/DAB

Approved:

SCOIL & MAIN ASH POND — STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN

Drawn: JJ

Date: NOVEMBER, 2007

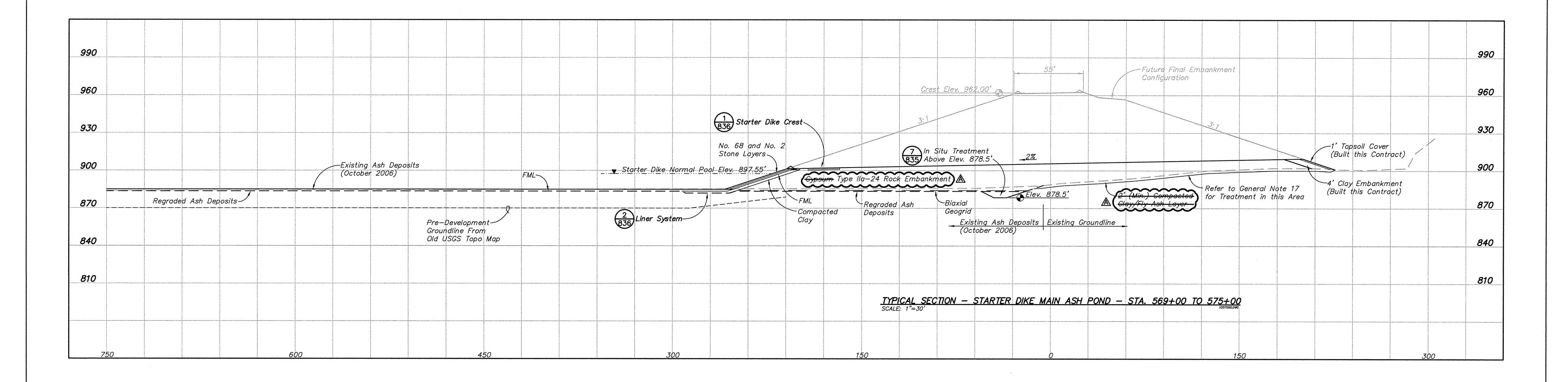
Chacked: VIS/DAB

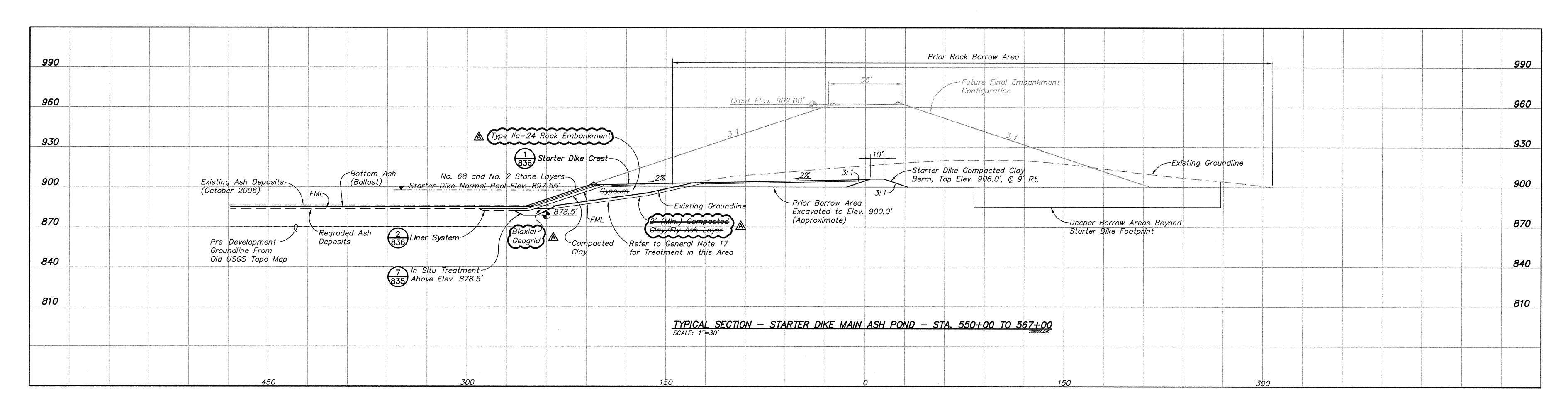
Approved:

JOB NO. JOB NO. JOB NO. JOB NO. Drawing No:

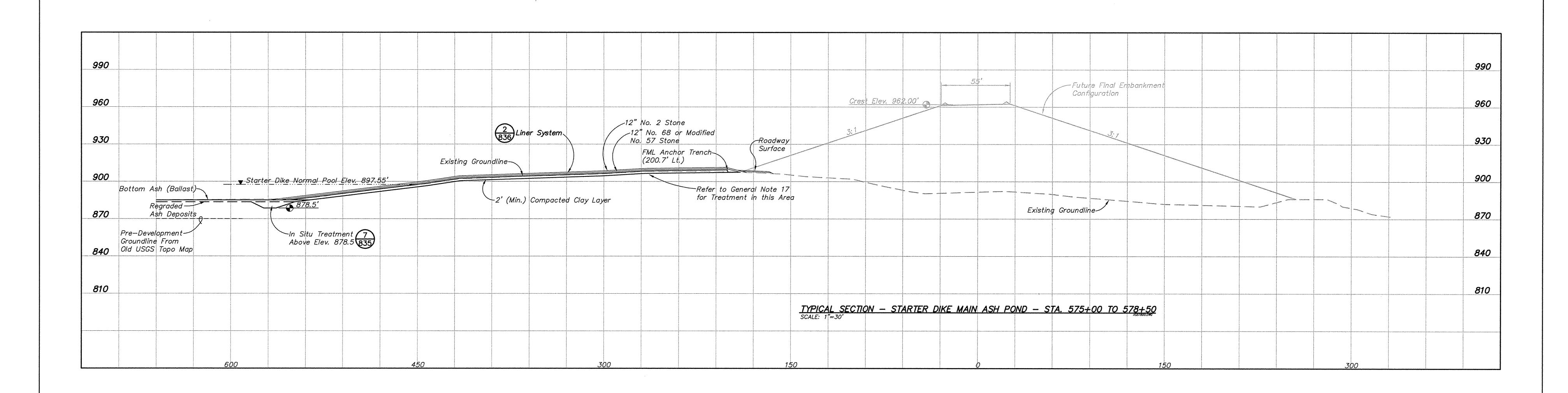
ENGINEERS

JOB NO. JOB NO. JOB NO. JOB NO. BRO—C—00831





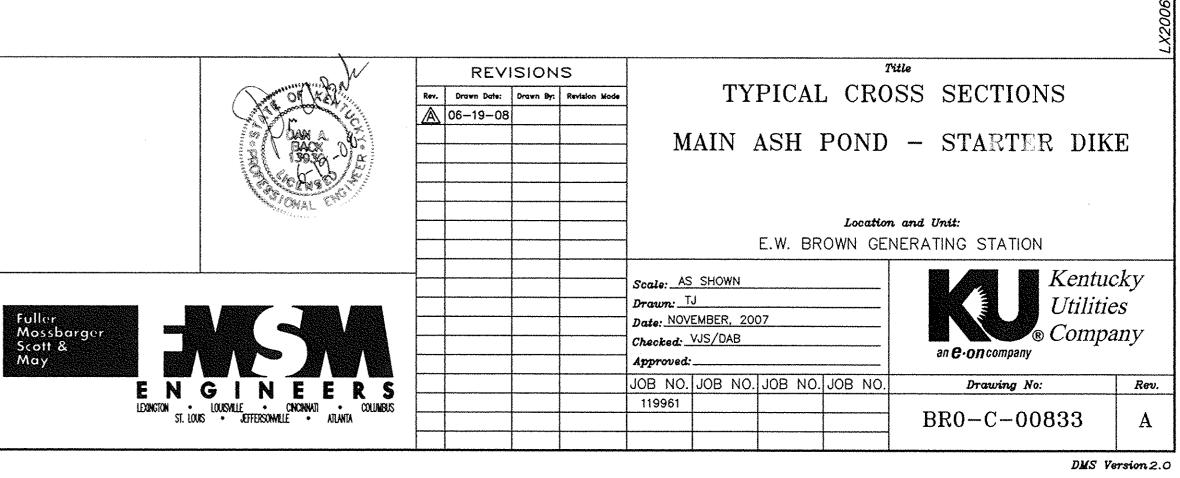
— Section or Detail No. Sheet Where Shown REFERENCE KEY

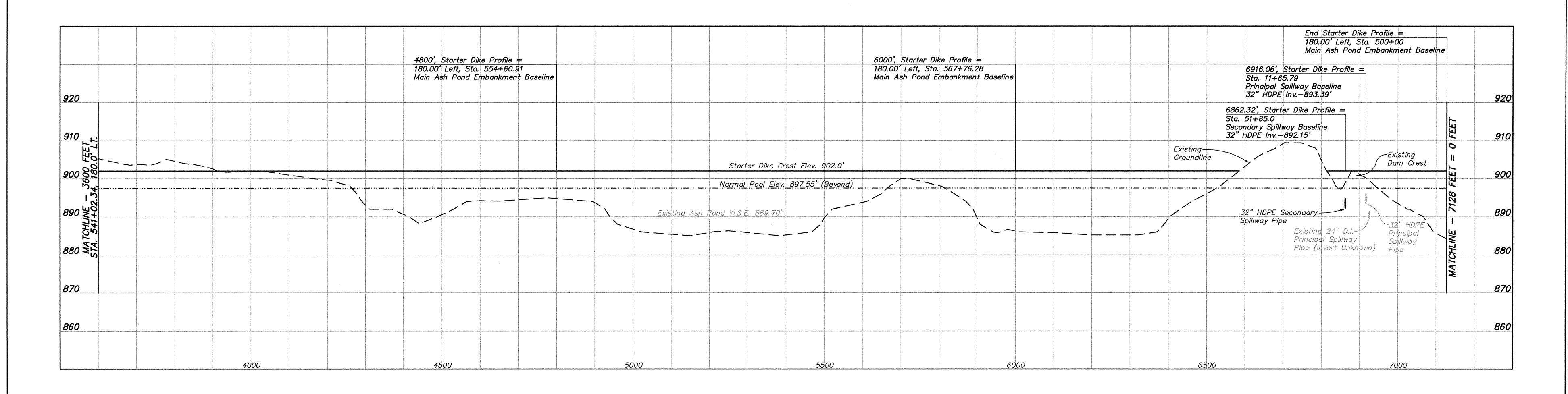


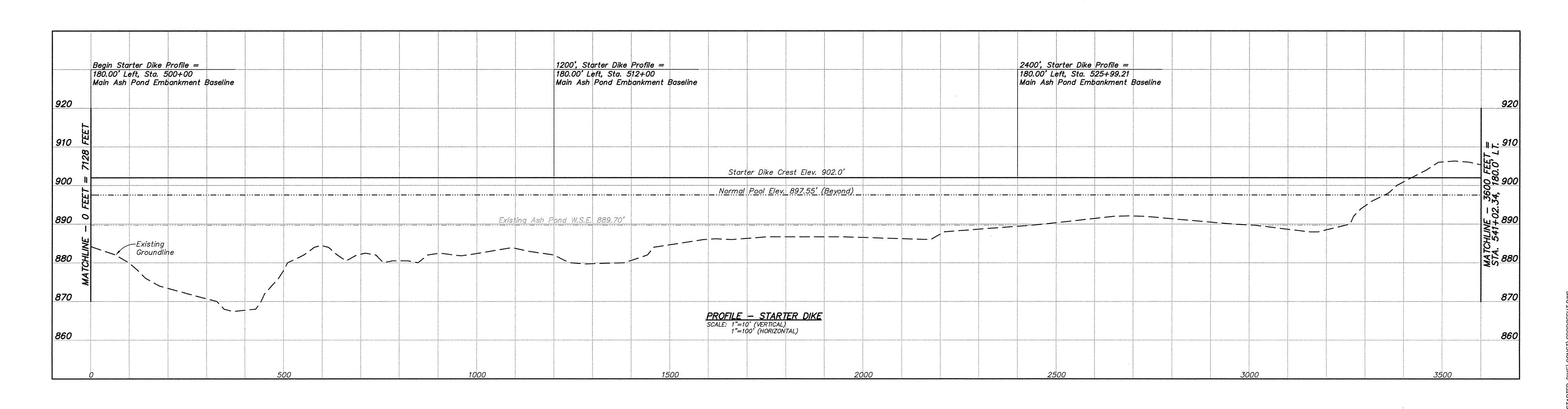
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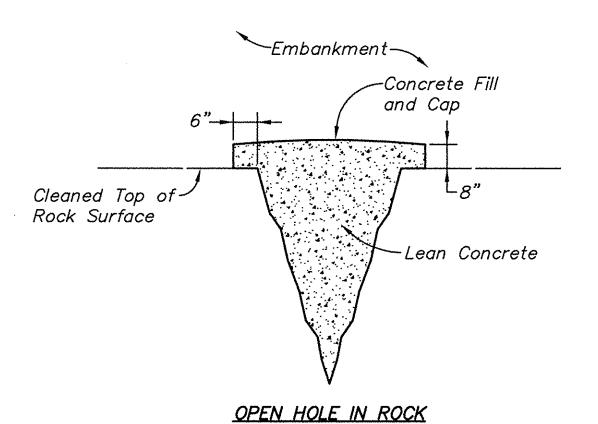
NOTE:

The existing groundline shown was taken from topographic and hydrographic mapping along the Starter Dike, 180' left of the Main Ash Pond Embankment Baseline. The Starter Dike profile is illustrated looking inward from outside of the pond.

OPENING IN ROCK, FILLED WITH NATURAL SOIL

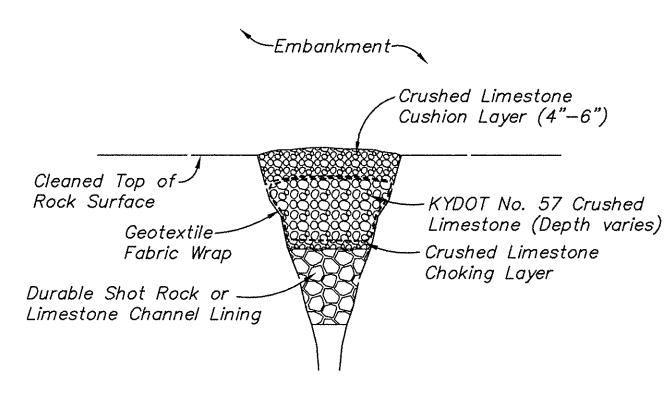
See Karst Feature Treatment Notes

1 DETAIL - IN SITU FOUNDATION TREATMENT 1A
835 NOT TO SCALE SEE SHEET 813



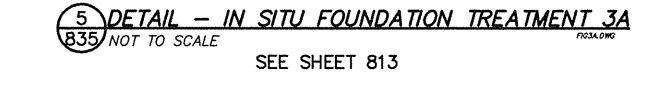
See Karst Feature Treatment Notes

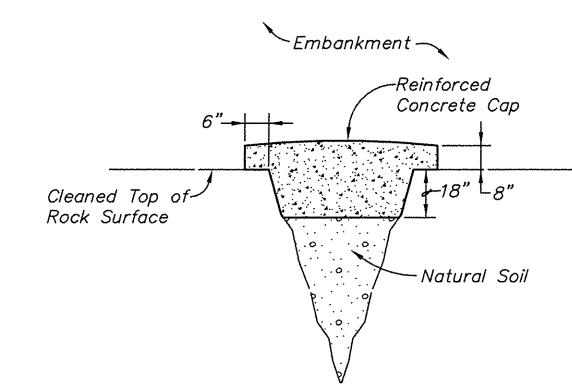
3 DETAIL — IN SITU FOUNDATION TREATMENT 2A
835 NOT TO SCALE SEE SHEET 813



NON-DRAINING, SOIL-FILLED CREVICE IN ROCK

See Karst Feature Treatment Notes

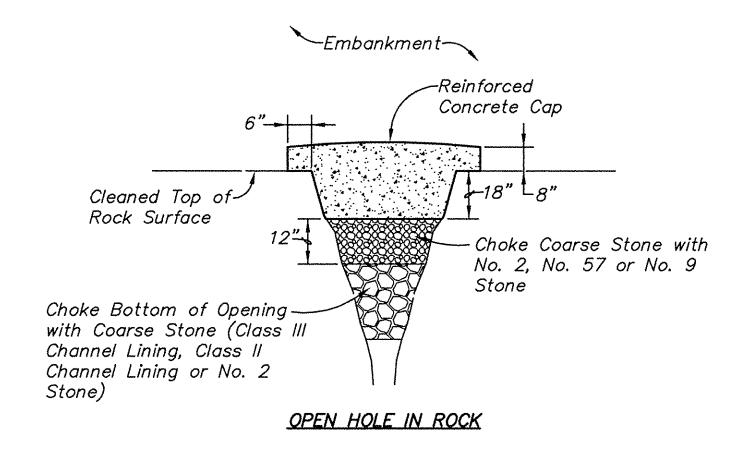




OPENING IN ROCK, FILLED WITH NATURAL SOIL

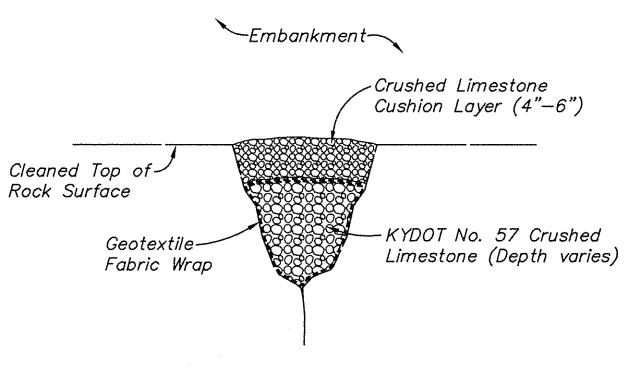
See Karst Feature Treatment Notes

2 DETAIL - IN SITU FOUNDATION TREATMENT 1B 835 NOT TO SCALE SEE SHEET 813



See Karst Feature Treatment Notes

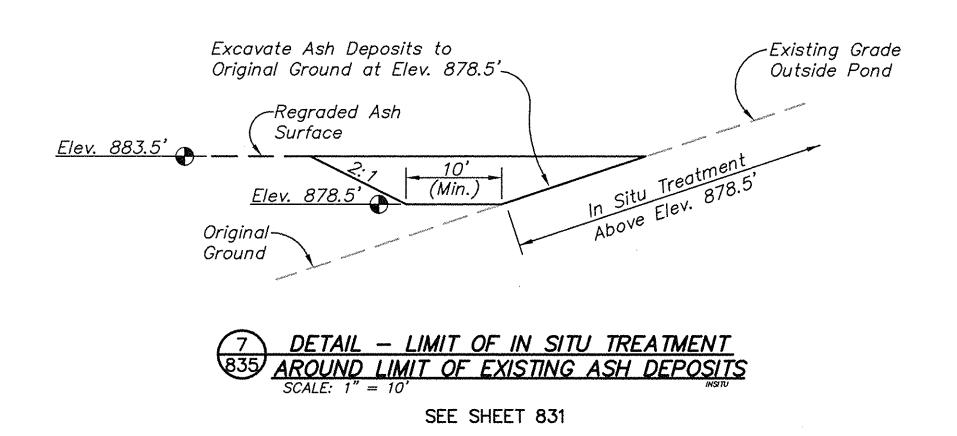
4 DETAIL - IN SITU FOUNDATION TREATMENT 2B 835 NOT TO SCALE SEE SHEET 813



NON-DRAINING, SOIL-FILLED CREVICE IN ROCK

See Karst Feature Treatment Notes

6 DETAIL — IN SITU FOUNDATION TREATMENT 3B 835 NOT TO SCALE SEE SHEET 813



KARST FEATURE TREATMENT NOTES

- 1. Project Site Karst The E. W. Brown Generating Station project site is known to contain karst activity and it is likely that solution type karst features (sinks, crevices, vertical joints, pinnacles, ridges and cavities) exist within the Main Pond Starter Dike embankment and pond liner foundation areas. The presence of these features could lead to uncontrolled movement of groundwater, which might in turn lead to movement of soil or ash material resulting in loss of structural foundation support and/or unacceptable discharges of contaminate laden surface water. The presence of the karst features also result in extremely differing bearing conditions for any overlying elements such as embankment or the pond liner section. The footprint area of the Starter Dike embankment and pond liner system shall be excavated to bedrock to the limits shown on plan sheets. Any karst features identified within these limits shall be treated in accordance with the above karst feature treatment details or as directed by the Owner's Representative.
- 2. <u>Top of Rock Cleaning</u> To permit the Owner's Representative to observe karst features and establish methods of treatment, the contractor shall clean the bedrock surface. The foundation area bedrock surface shall be exposed by removal of all soil which can be excavated with a maximum size eighteen (18") inch excavator/backhoe bucket. Final scraping across the top of the bedrock shall result in a generally clean and exposed rock surface, however, in isolated areas, up to two (2) inches of soil like material may be accepted remaining on the bedrock surface, provided the Owner's Representative can sufficiently observe the foundation area for any karst features. The excavator bucket shall be dragged across the entire rock surface in the presence of the Owner's Representative and the top of bedrock cleaning shall be completed to the satisfaction of the Owner's Representative. When so directed by the Owner's Representative, laborers shall also be employed to remove loose rocks and soil material from the top of rock surface. Where deep crevices exist in the top of rock surface, the Contractor shall use available equipment to remove any loose rock and soil material from the top and sides of the crevice to allow the safe entry of personnel into the lower top of rock areas. Surface grading around top of rock cleaning areas shall be performed (and maintained) in a manner to preclude the entry of significant surface runoff. Where significant groundwater is entering the top of rock feature being cleaned and treated, the contractor shall utilize pumps and, if needed, soil berms to maintain a generally dry area for evaluation and treatment.
- 3. <u>Treatment</u> The contractor shall treat karst features identified by the Owner's Representative. Each karst feature will have different and unique characteristics and the treatment method will be developed to address the conditions of the feature once they have been exposed and evaluated. Treatment shall be as directed by the Owner's Representative. Treatment may be according to these details or other methods which the Owner's Representative establishes.
- 4. Notification Generally, the appropriate Owner's Representative will be on site during excavation and embankment construction. The Contractor shall, however, give at least a two work day formal notice to the designated Owner's Representative prior to anticipated top of rock and karst features inspection to allow the appropriate individual to be present.
- 5. Payment Terms Payment for excavation to the top of rock shall be in accordance with the Contract specification requirements for excavation. As indicated in the specifications, this excavation shall be considered as mass yardage, up to the point at which an excavator (Cat 325 or equal) and a standard tandem axle highway dump truck cannot reasonably access the excavation. This excavation shall be considered incidental to the embankment in-place price for clay stockpile, clay embankment and/or waste embankment, as applicable. Payment will be made on a measured or plan yardage basis at the Contract unit rates for that type of embankment. Any additional soil excavation, top of rock cleaning, karst feature treatment, and initial backfilling will be measured and paid on a time and materials basis. The quantities of materials, including lean concrete, steel reinforced concrete, and various stone applications, will be determined as karst treatment progresses. Material usage and treatment may not be limited to the previously mentioned items, but shall be as instructed by the Owner's Representative. Payment for materials will be based on actual delivery ticket prices/quantities for these materials and actual invoices. Payment will be actual cost plus agreed markup. Karst treatment equipment, contractor manpower to operate the equipment, and labor applicable to the karst treatment will paid for at the unit rates provided in the Contract. The actual equipment and operator costs shall be combined into one unit cost for each type of equipment listed. All equipment and materials not listed on the project bid form will considered incidental to karst treatment construction and will not be considered for payment. The Contractor shall use all reasonable diligence to execute work, performed on a time and materials basis, in an efficient and expeditious manner, providing the Owner maximum value for the resources expended, consistent with the requirements and directives relative to the treatment process. This shall include all labor, equipment and materials directed for use in the in situ treatment process. Backfilling, following completion of the specific in situ treatment items, shall be in accordance with specification requirements for the applicable zone of embankment. Where a power compactor cannot reasonably reach the area being filled, payment shall be on the basis of yardage at the Contract trench backfill unit rate. All other backfill shall be measured on a yardage basis and paid at the Contract unit rate for that type of embankment.

Volumes for payment for stockpile and waste material will be computed from survey data collected by the Contractor's surveyor. As a minimum, the Contractor's surveyor shall perform surveys on the stock pile and waste areas immediately prior to initial excavation, at the point when the Owner's Representative and Contractor agree that further excavation should be performed on a time and materials basis, at the point when the non—in situ backfilling begins (the point when the Owner's Representative and Contractor agree that further filling should be performed as typical embankment yardage). The final grade for volume calculations shall be based on the Plan configuration, not a final survey, unless the Owner's Representative directs a different final configuration be completed. Each field survey and all volume calculations completed by the Contractor's surveyor shall be submitted to and reviewed by the Owner's Representative. Surveying shall be incidental to embankment and no separate payment shall be made.

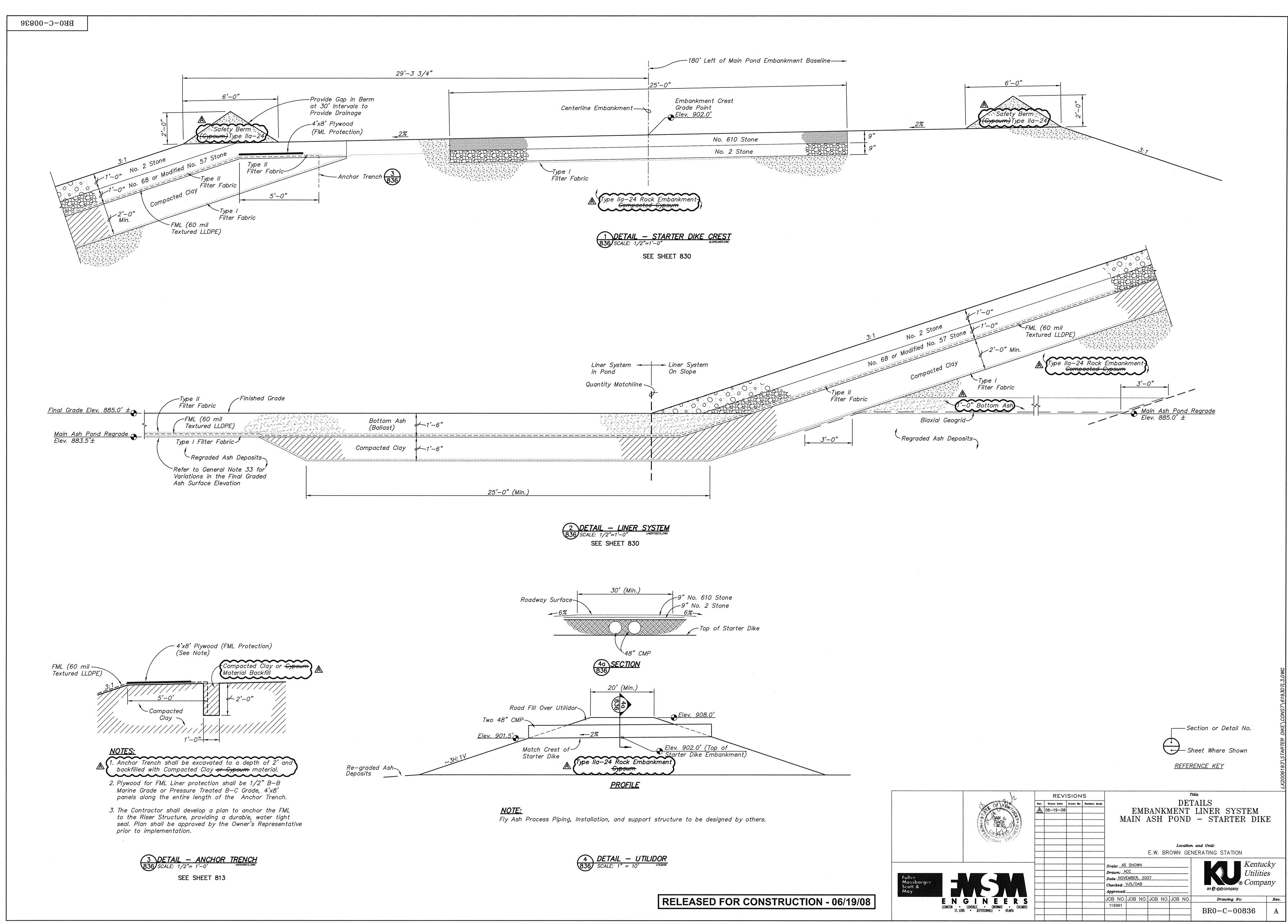
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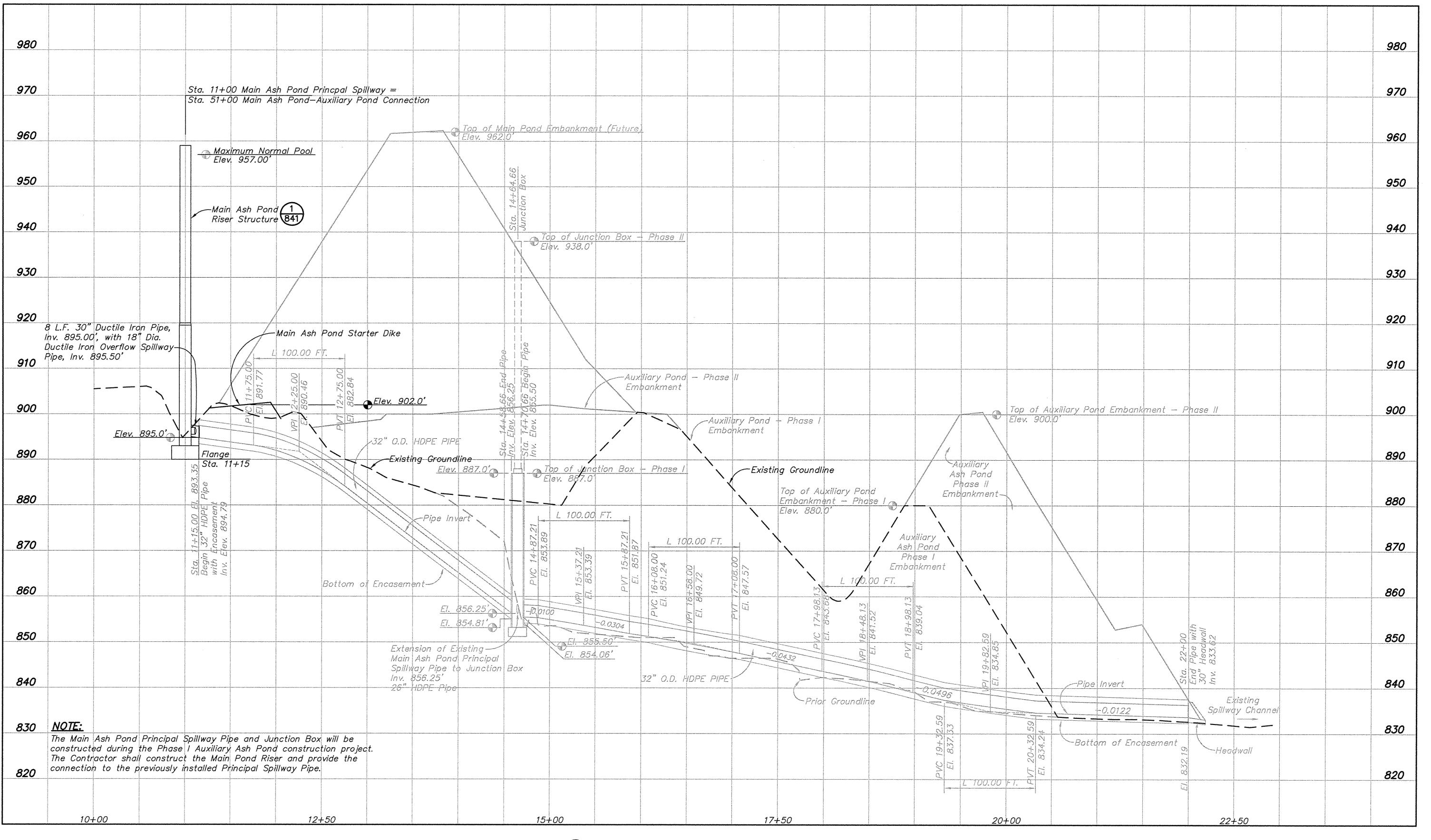
REFERENCE KEY

Drawing No:

BR0-C-00835

REVISIONS DETAILS Rev. Drawn Date: Drawn By: Revision Made FOUNDATION TREATMENT MAIN ASH POND - STARTER DIKE E.W. BROWN GENERATING STATION Date: NOVEMBER, 2007 Checked: VJS/DAB an **e·on** company JOB NO. JOB NO. JOB NO. JOB NO.





**NOTE:**Existing pipe geometry and prior existing groundline provided by Bizzack Construction.

1 PROFILE - MAIN ASH POND PRINCIPAL SPILLWAY
839 SCALE: 1"=10' (VERTICAL)
1"=50' (HORIZONTAL)

Section or Detail No. Sheet Where Shown REFERENCE KEY

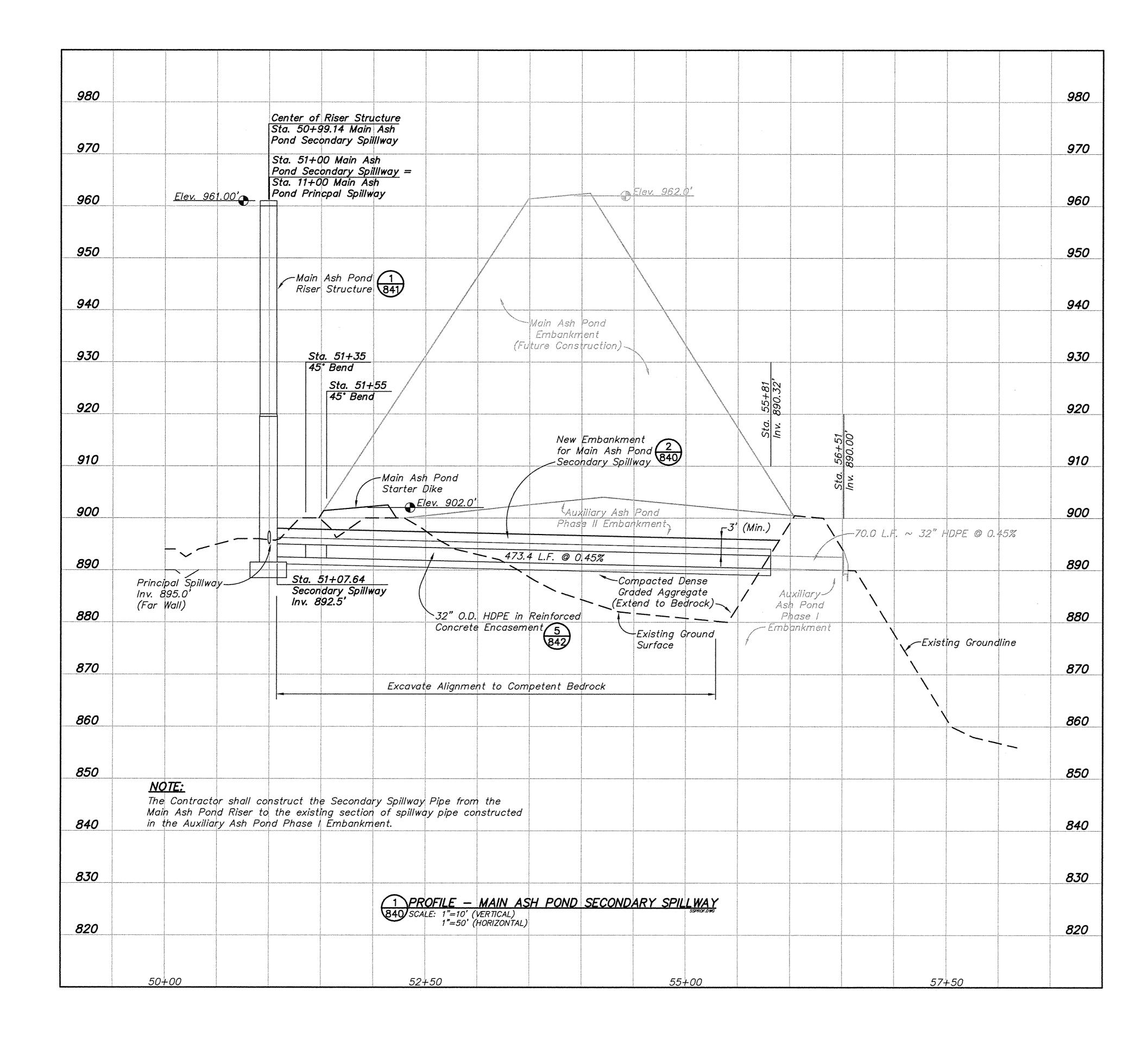
REVISIONS PROFILE - PRINCIPAL SPILLWAY Rev. Drawn Date: Drawn By: Revision Made MAIN ASH POND - STARTER DIKE

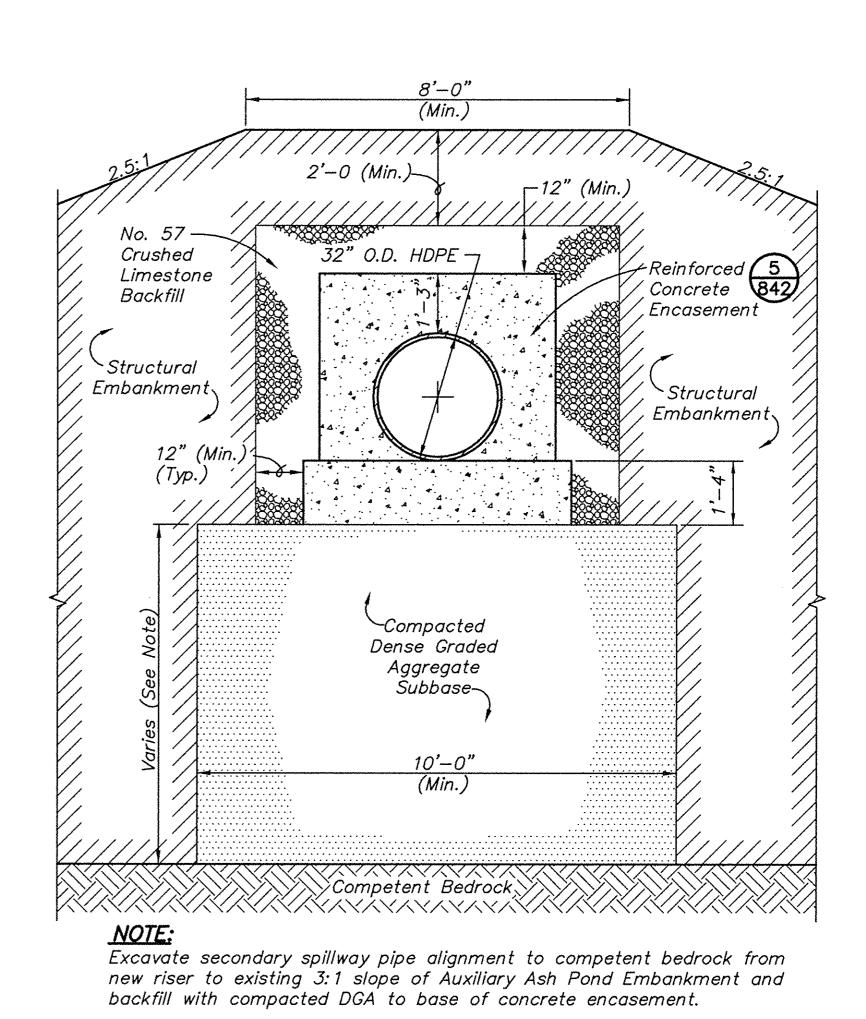
Location and Unit: E.W. BROWN GENERATING STATION

Scale: AS SHOWN Drawn: SLB/TJ Date: NOVEMBER, 2007 Checked: VJS/DAB Approved: \_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 BR0-C-00839

an **e·on** company Drawing No:

RELEASED FOR CONSTRUCTION - 06/19/08

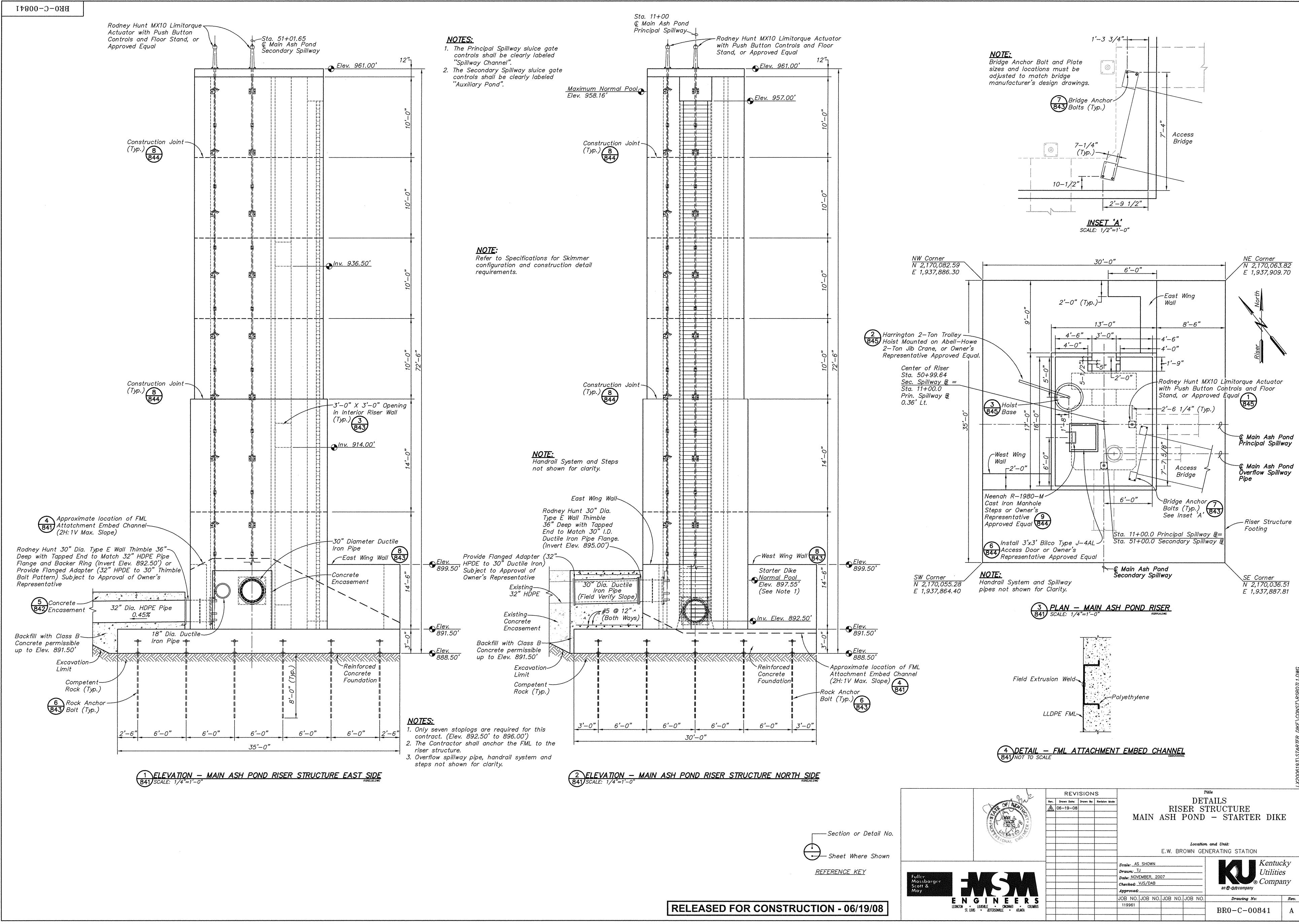


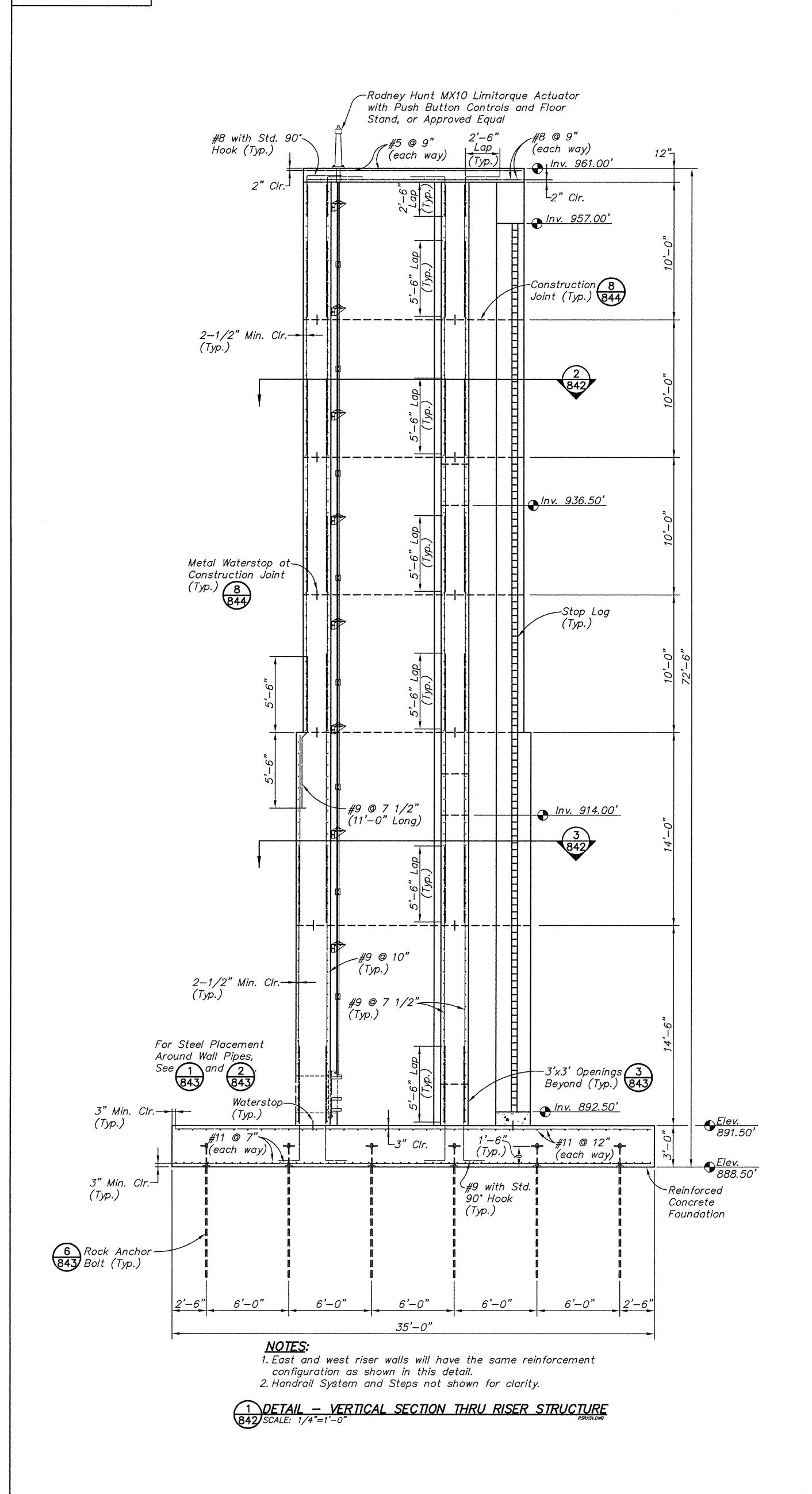


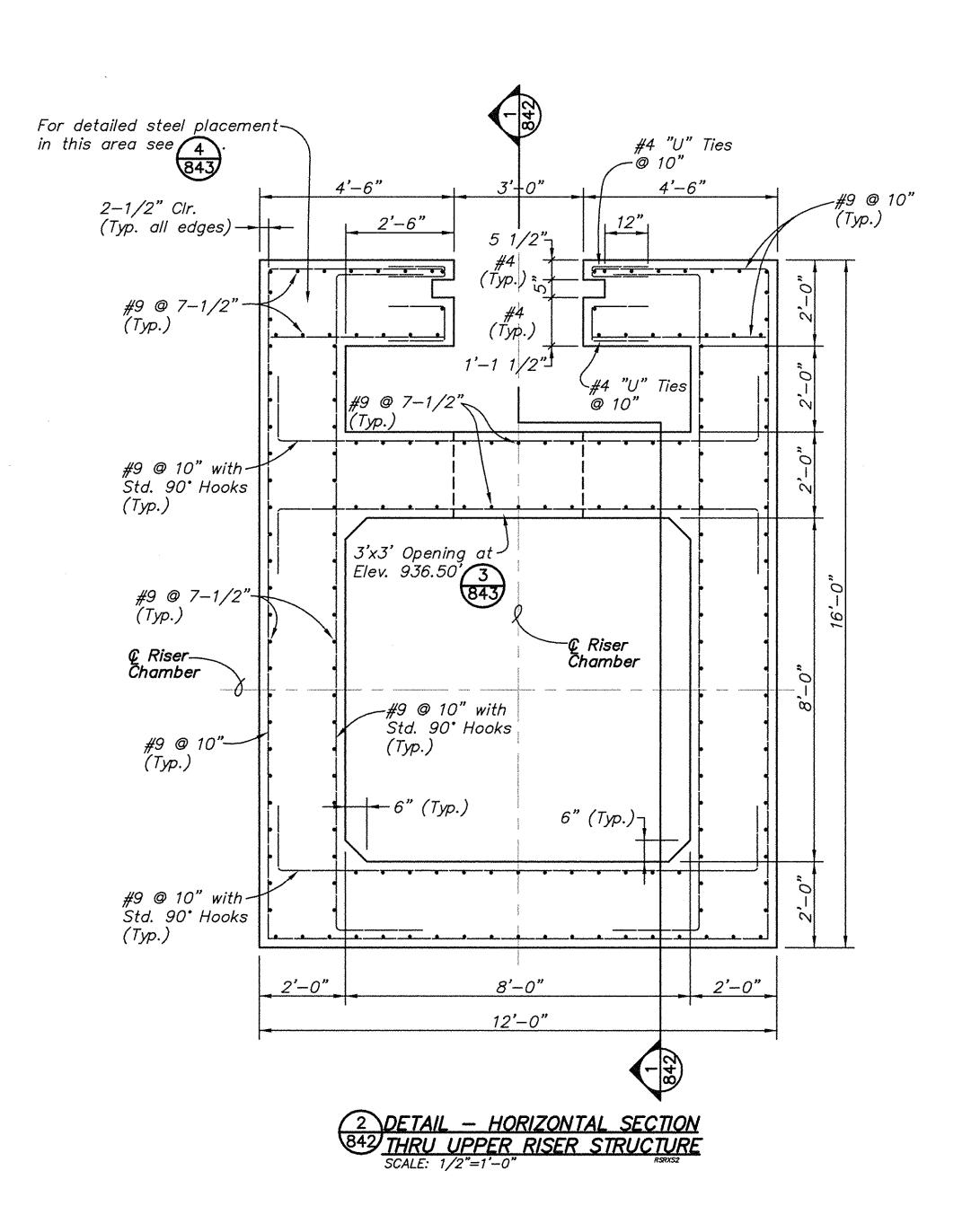
2 DETAIL - SECONDARY SPILLWAY EMBANKMENT INSTALLATION
840 SCALE: 1/2"=1'-0"
SECSPILLOWS

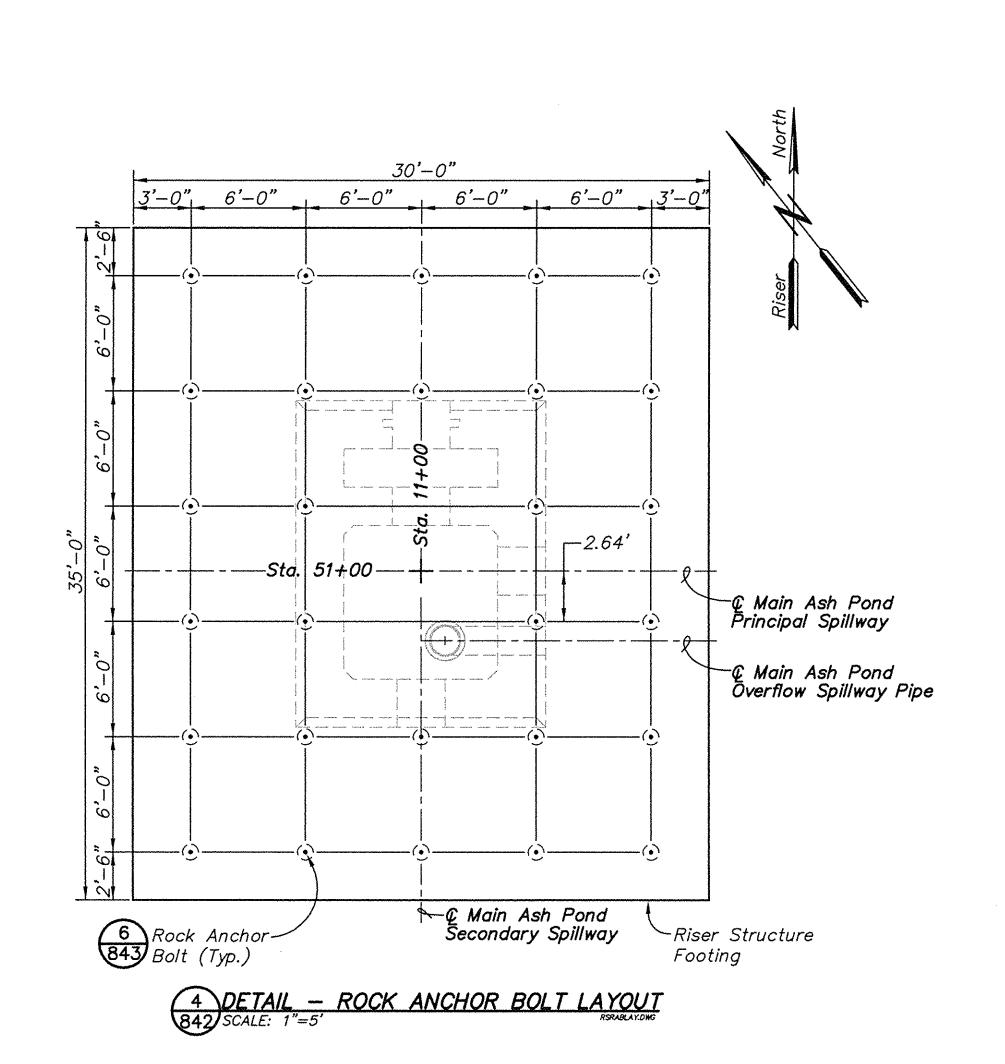
Section or Detail No. Sheet Where Shown REFERENCE KEY

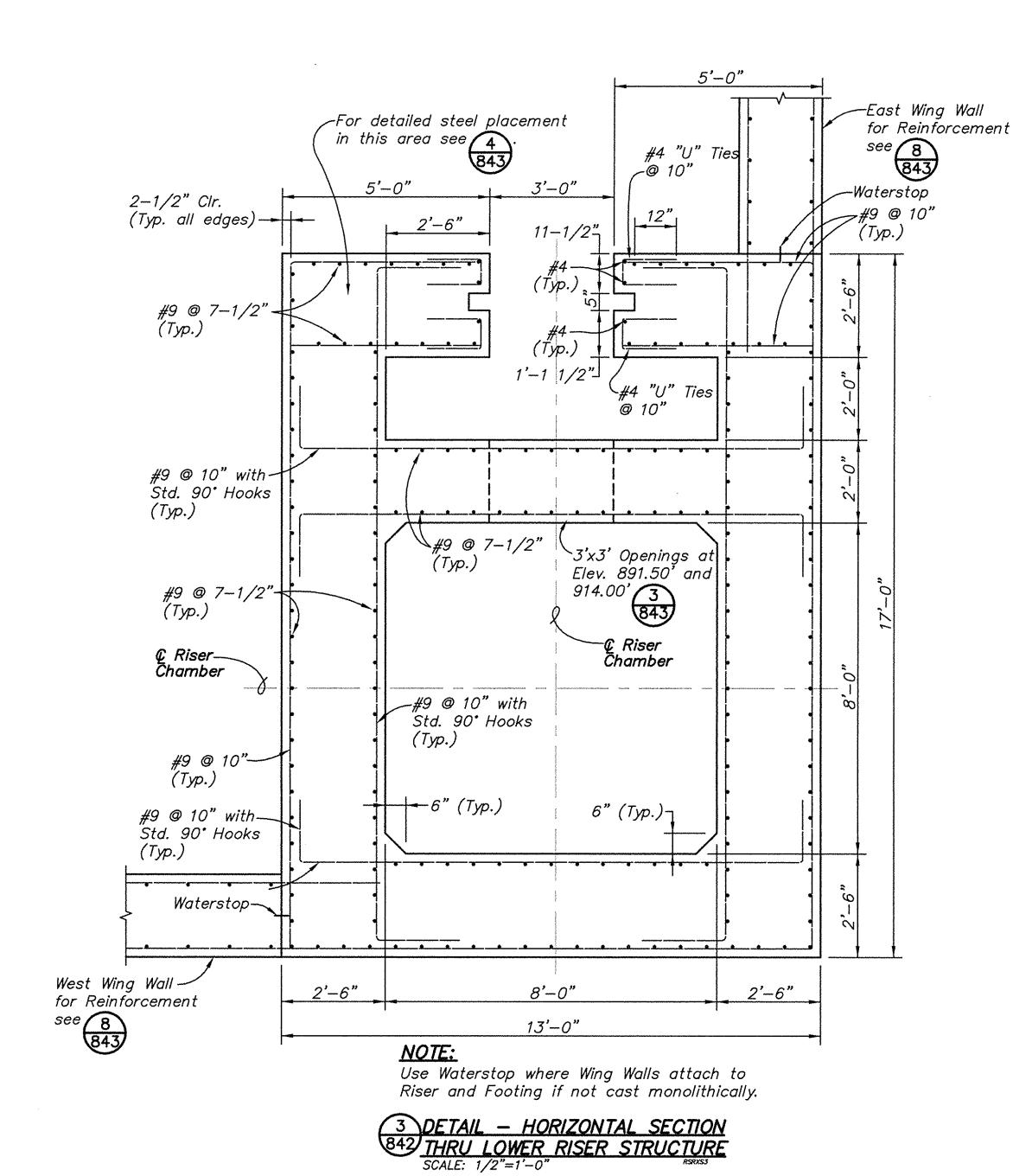
REVISIONS PROFILE - SECONDARY SPILLWAY MAIN ASH POND - STARTER DIKE Location and Unit: E.W. BROWN GENERATING STATION Drawn: TJ/ACC Date: NOVEMBER, 2007 Checked: VJS/DAB an **e-on** company Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 Drawing No: BR0-C-00840

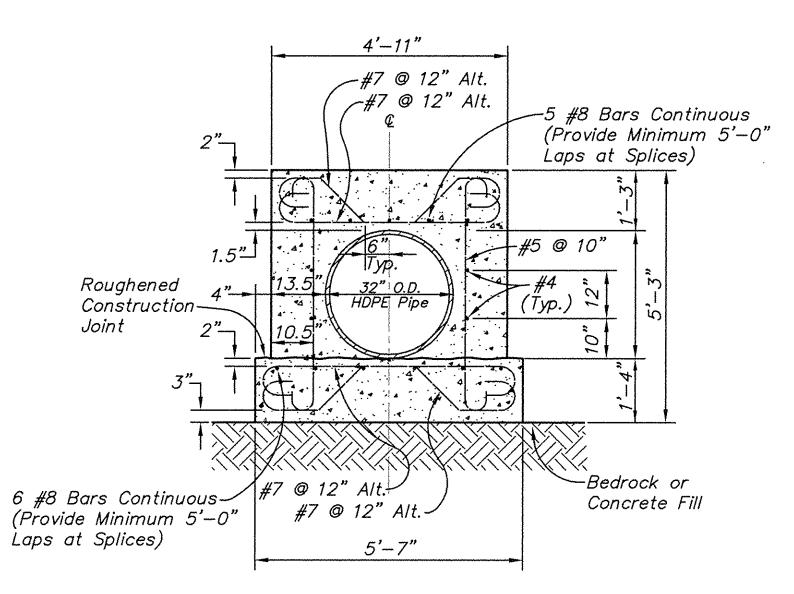












5 DETAIL - CONCRETE ENCASEMENT FOR 32" O.D. HDPE PIPE 842 SCALE: 1/2"=1'-0"

Sheet Where Shown

REFERENCE KEY

DETAILS
RISER STRUCTURE
MAIN ASH POND - STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Fuller
Mossbarger
Scott &
May

ENGINEE COUNTY:
LEXINGTON LOUISMILE COUNTY:
ST. LOUIS PEFERSONMILE ATLANTA

GENERATING STATION

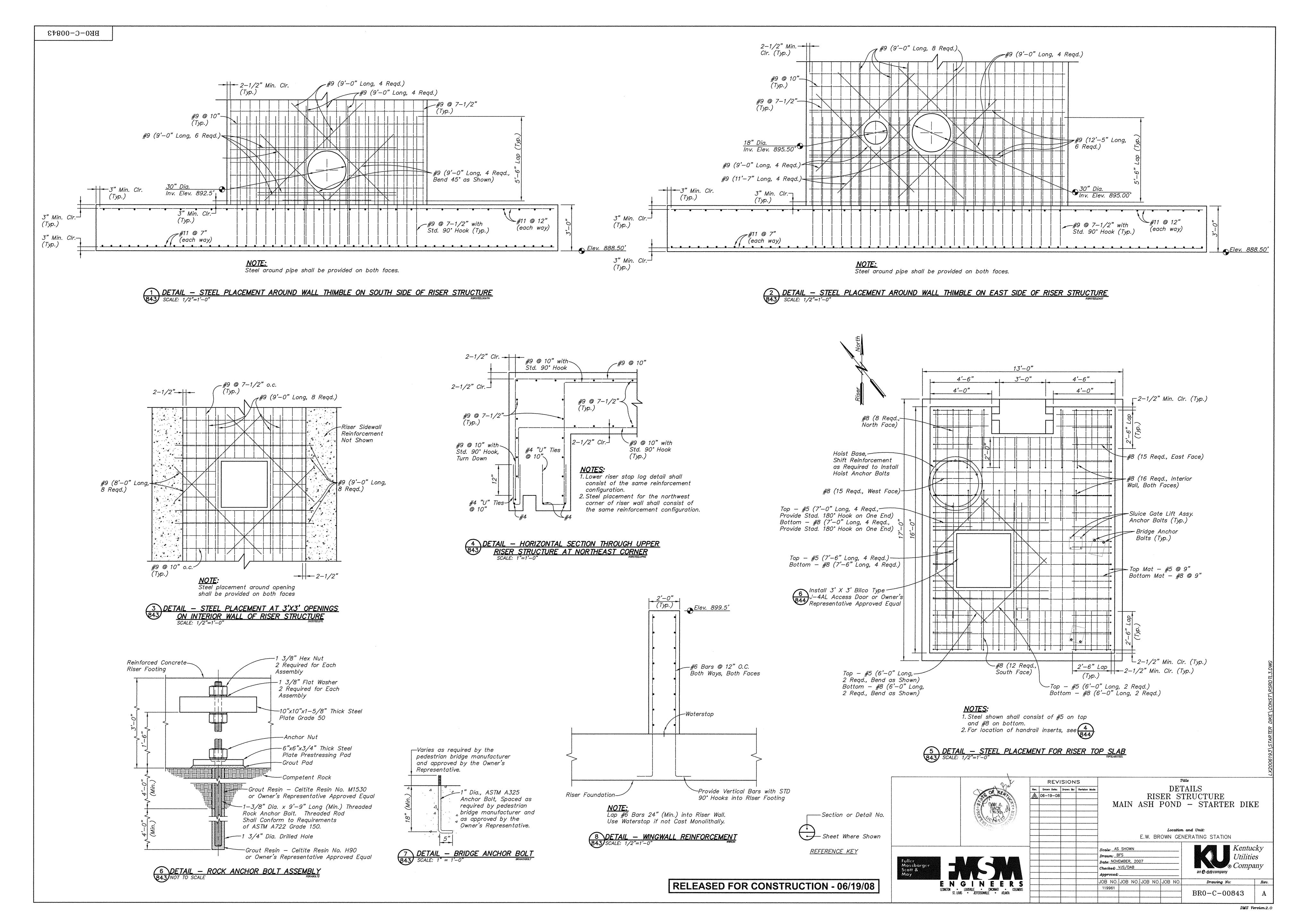
Kentucky
Utilities

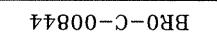
© Company

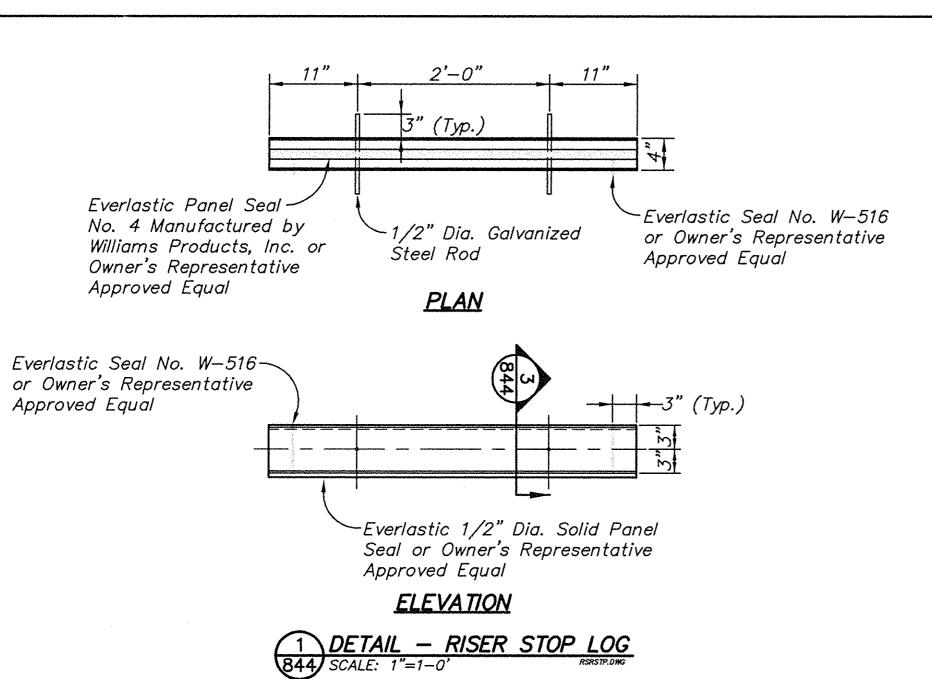
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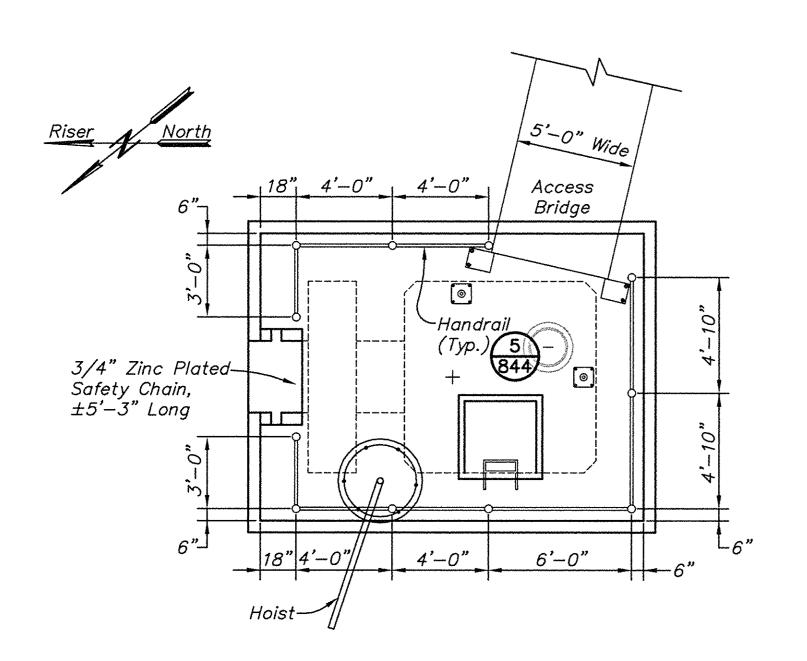
BR0-C-00842

RELEASED FOR CONSTRUCTION - 06/19/08

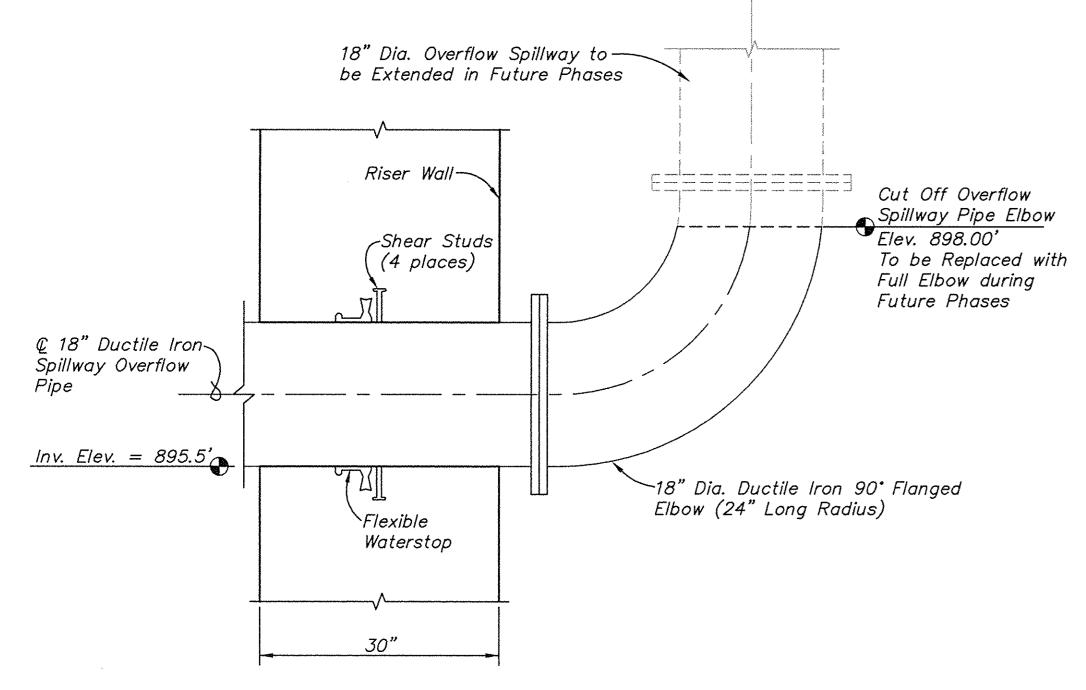






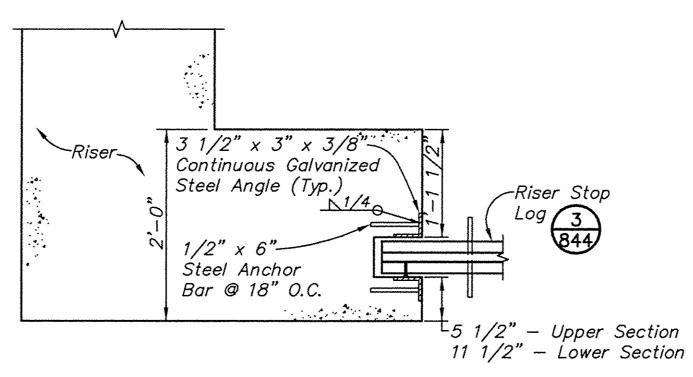


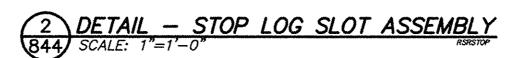


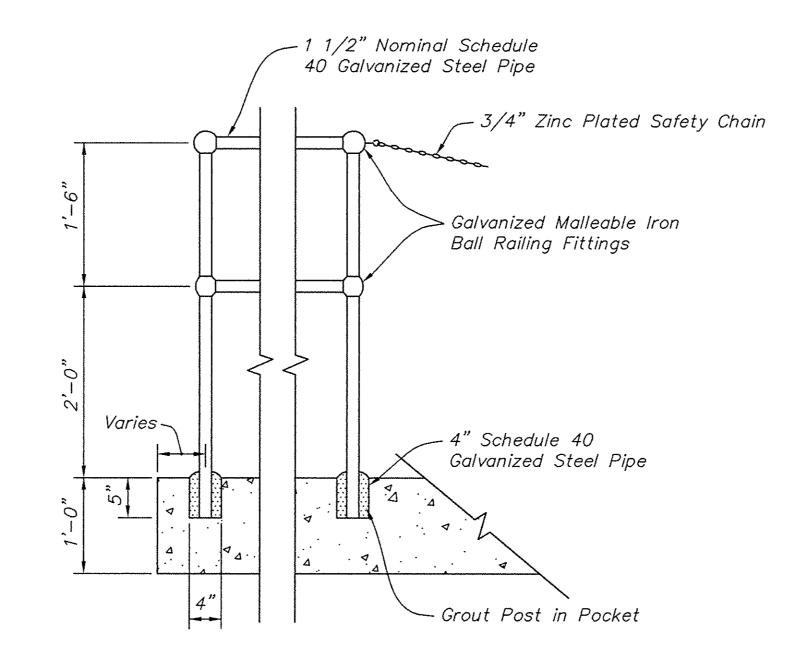


- 1. Steel pipe shall be cast into concrete.
- 2. Shear resistant studs shall be 4 standard 3/4" diameter by 4" long and conform to AWS D.1.1. Studs shall be located equal distant apart such that they fall in the
- center of the wall. 3. Flexible waterstop shall be waterstop (WS-30) as manufactured by Press-seal Gasket Corporation, or approved equal. The waterstop shall consist of a rubber gasket and one external take-up clamp. Limit of gasket shall be a minimum 1.5-inches inside face of concrete wall.





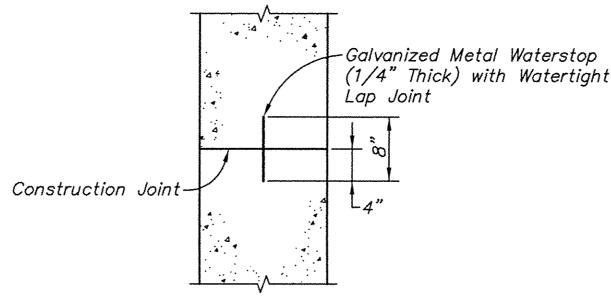




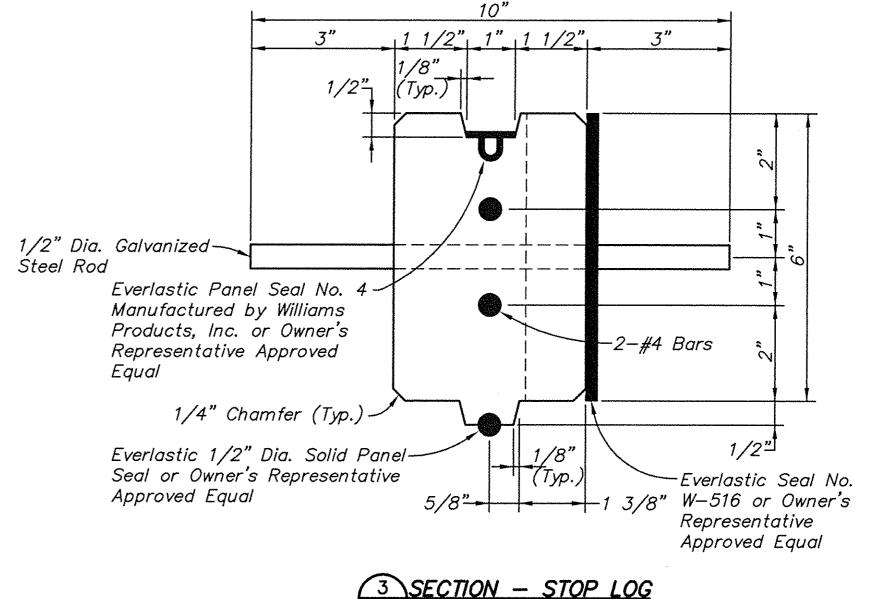
5 SECTION - TYPICAL INSTALLATION OF HANDRAIL

844 Scale: 1"=1'-0"

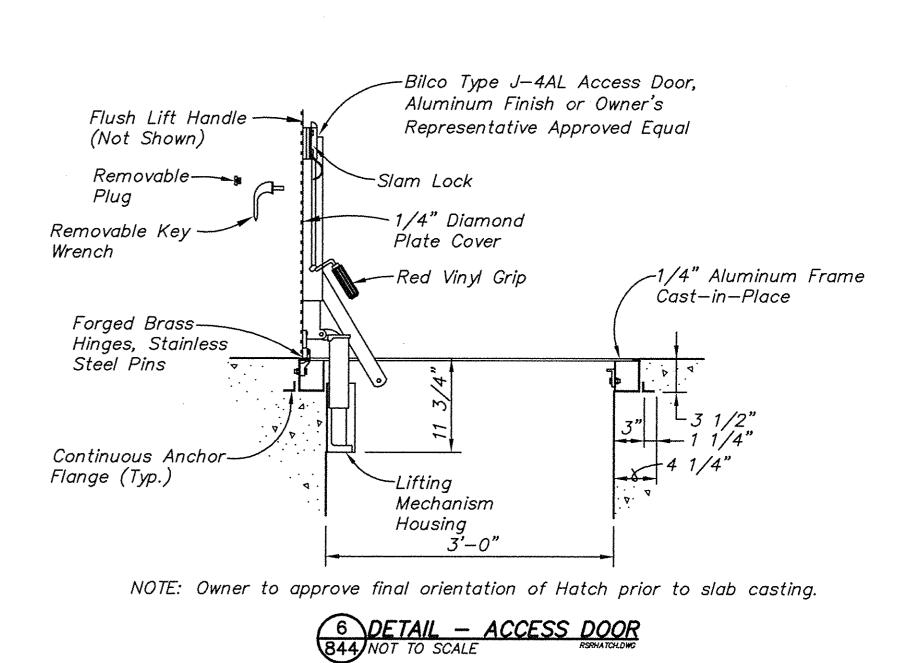
HRALELY,DIRG

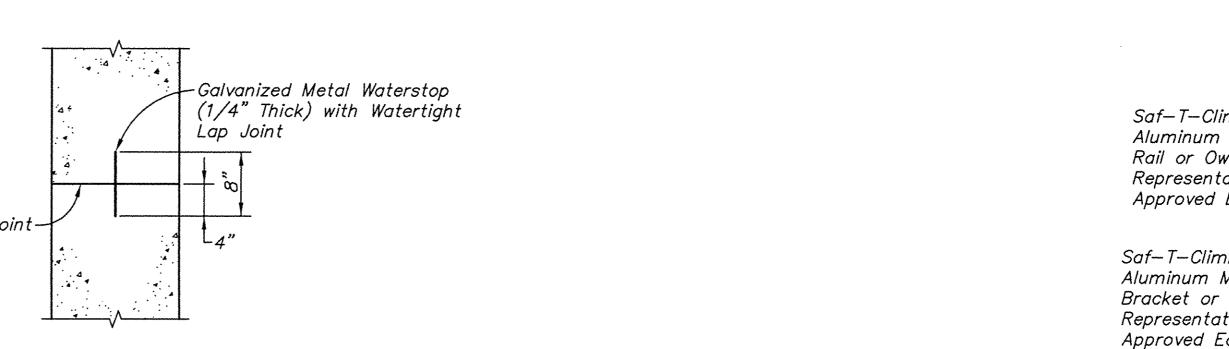


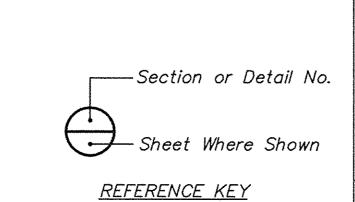
8 DETAIL - CONSTRUCTION JOINT AND
METAL WATERSTOP IN RISER WALL
SCALE: 1"=1'-0"
RSRWATSP.DIRG



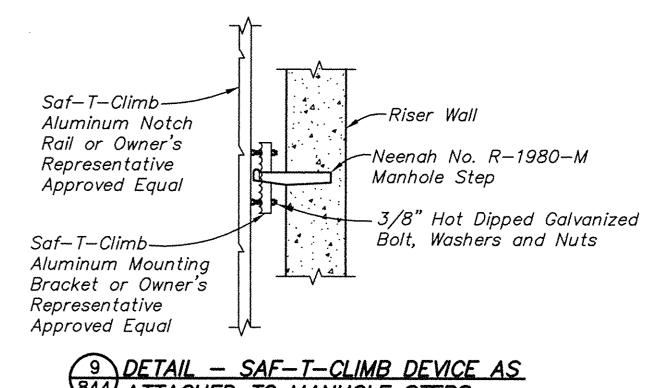
 $\frac{3}{844} \frac{SECTION - STOP \ LOG}{1/2" = 1'-0"}$ 

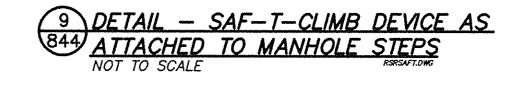


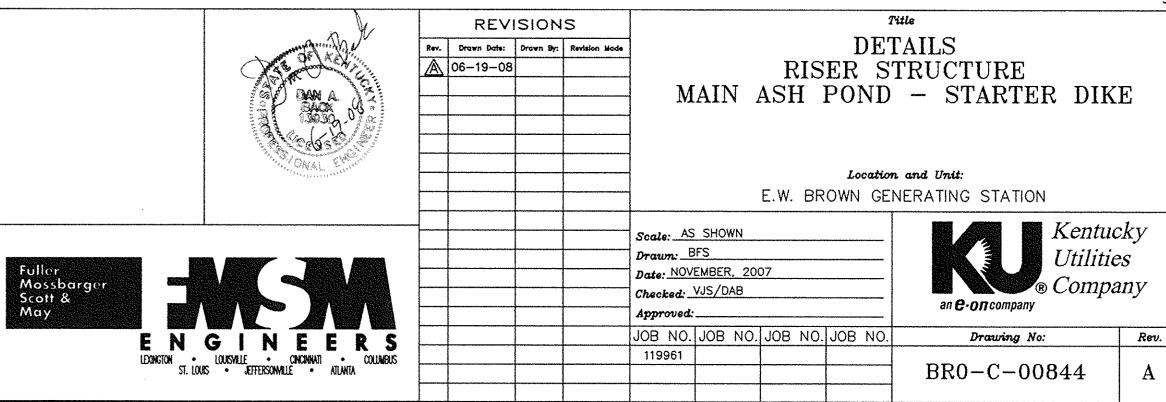


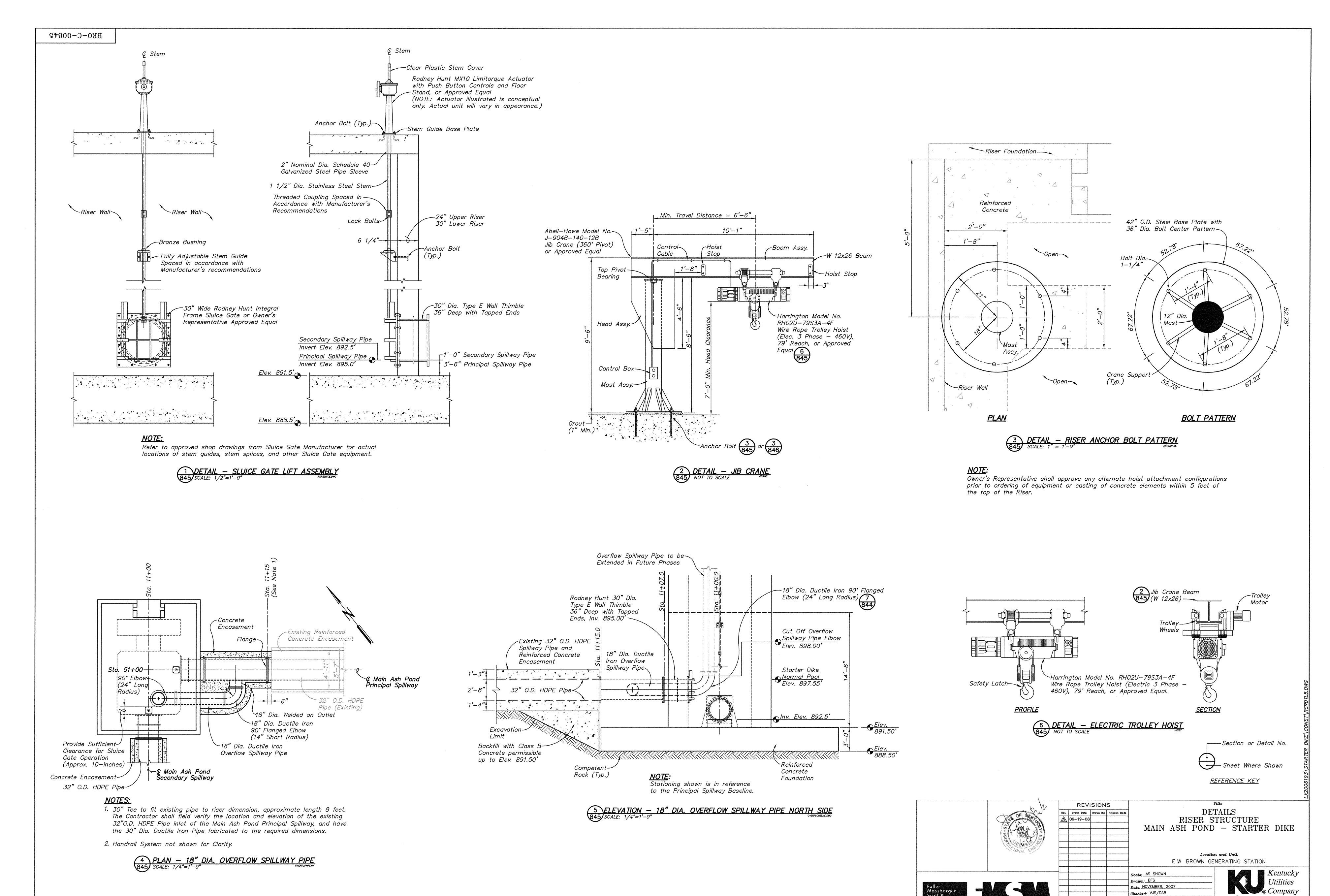












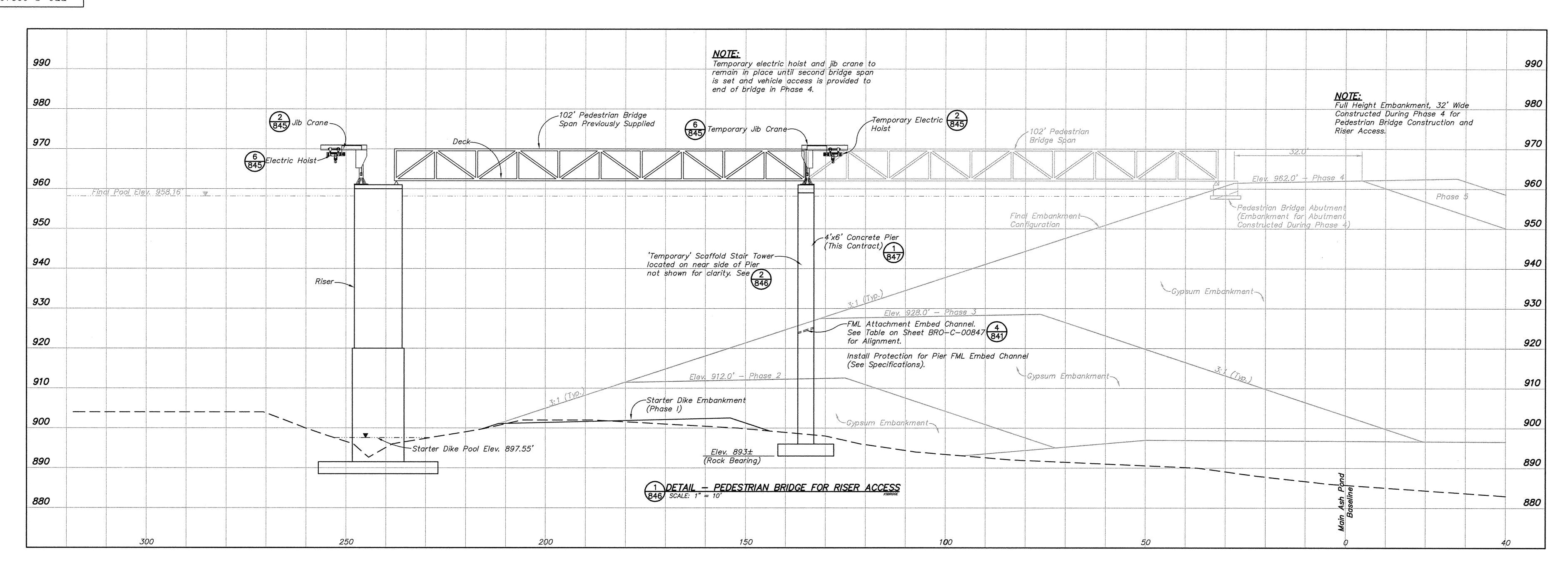
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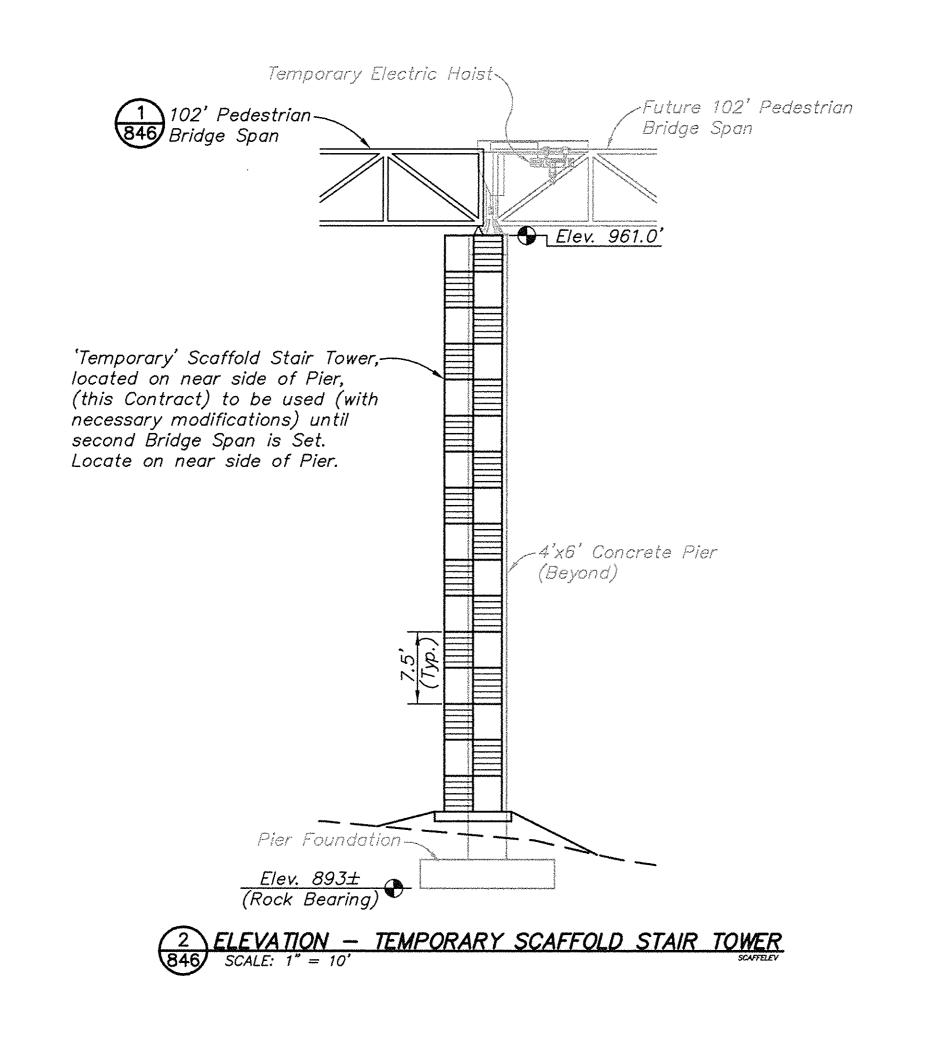
DMS Version 2.0

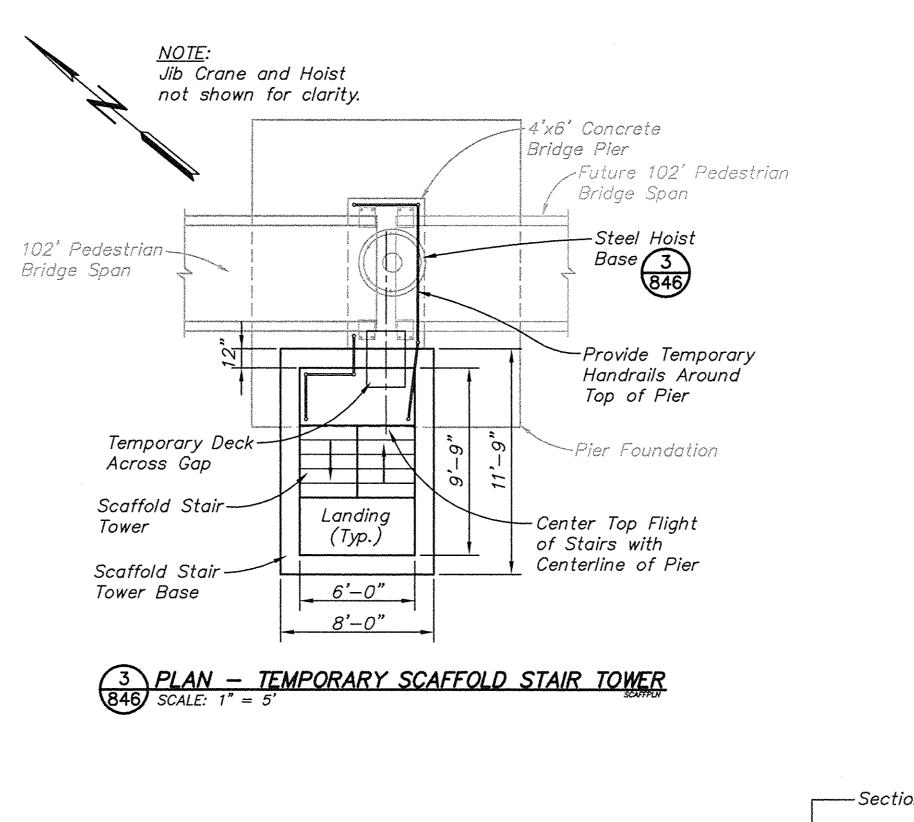
Drawing No:

BR0-C-00845

JOB NO. JOB NO. JOB NO. JOB NO.

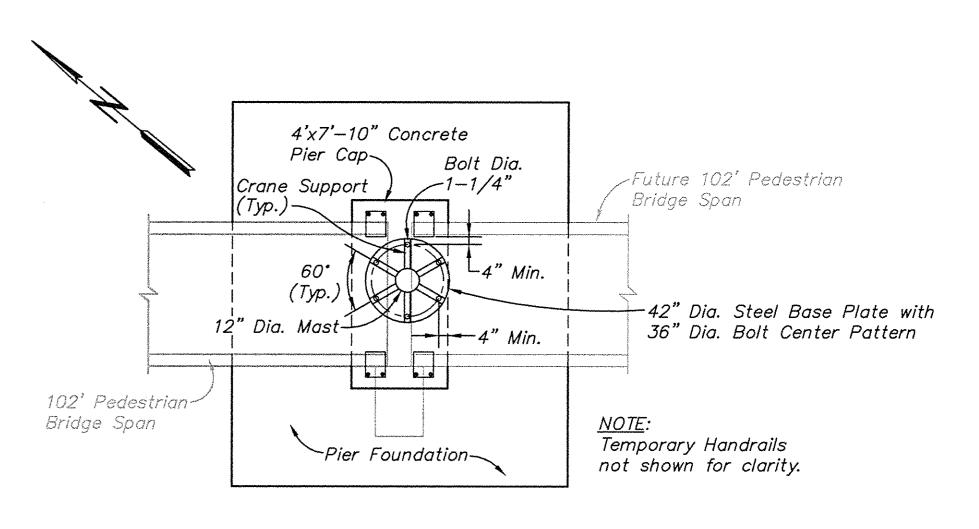






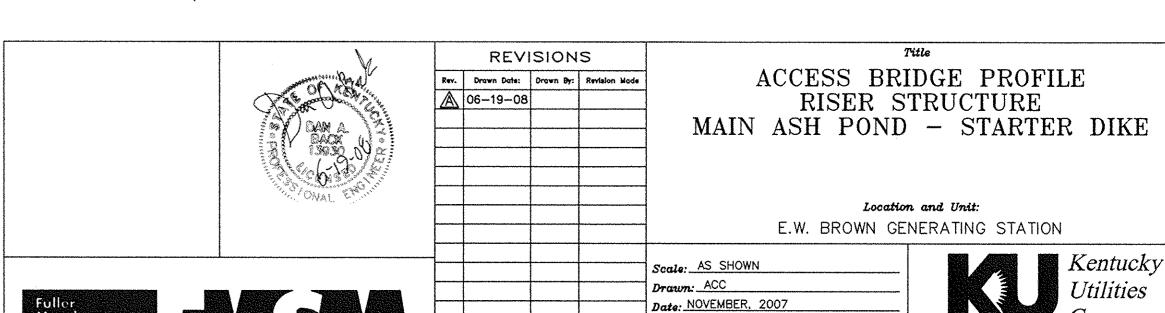
——Section or Detail No. Sheet Where Shown REFERENCE KEY

RELEASED FOR CONSTRUCTION - 06/19/08



3 DETAIL - BRIDGE PIER ANCHOR BOLT PATTERN
846 SCALE: 1/4" = 1'-0"
BPANOHDATS

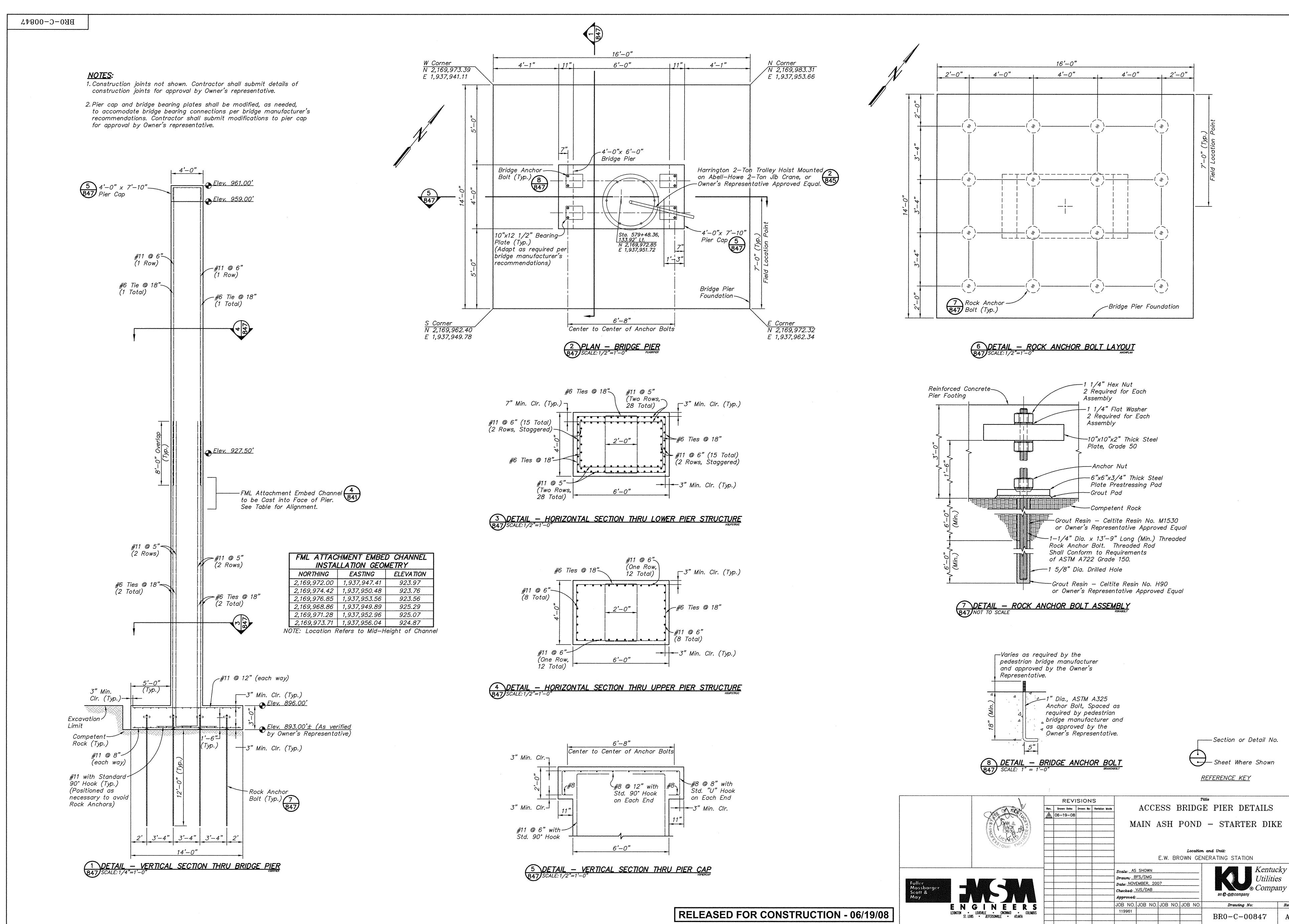
Owner's Representative shall approve any alternate hoist attachment configurations prior to ordering of equipment or casting of concrete elements within 5 feet of the top of the Riser.



E.W. BROWN GENERATING STATION Date: NOVEMBER, 2007 Checked: VJS/DAB an **e-on** company JOB NO. JOB NO. JOB NO. JOB NO. Rev.

Drawing No: BR0-C-00846

DMS Version 2.0



BB0-C-00821 Top of Dam Elev. = 962.0' Main Ash Pond Normal Pool El. 958.2' 950 H 930 § *920* Top of Starter Dike = 902.0 900 Starter Dike Normal Pool El. 897.6 *890* 2000 *3000* STORAGE (ACRE-FEET) AREA - CAPACITY CURVE *3000* 2500 Final Configuration Inflow Freeboard Hydrograph 2000 Starter Dike Inflow Freeboard Hydrograph 1500 °1000 500 DESIGN HYDROGRAPHS

AREA (ACRES)

4000

Starter Dike Outflow Freeboard Hydrograph

TIME (HOURS)

*5000* 

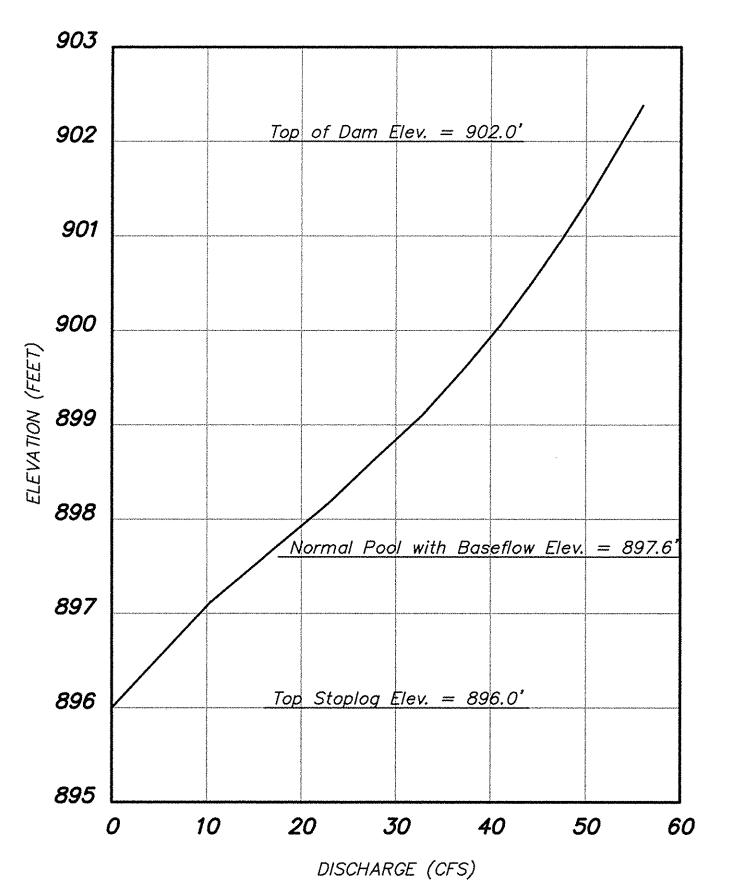
6000

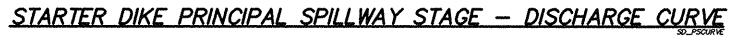
Final Configuration Outflow

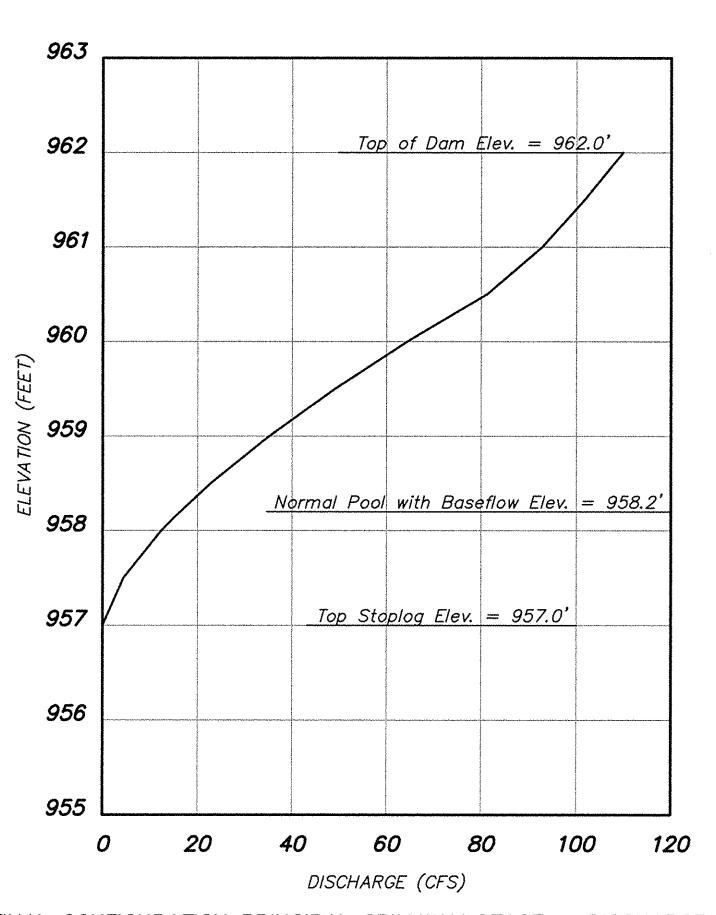
Freeboard Hydrograph

50

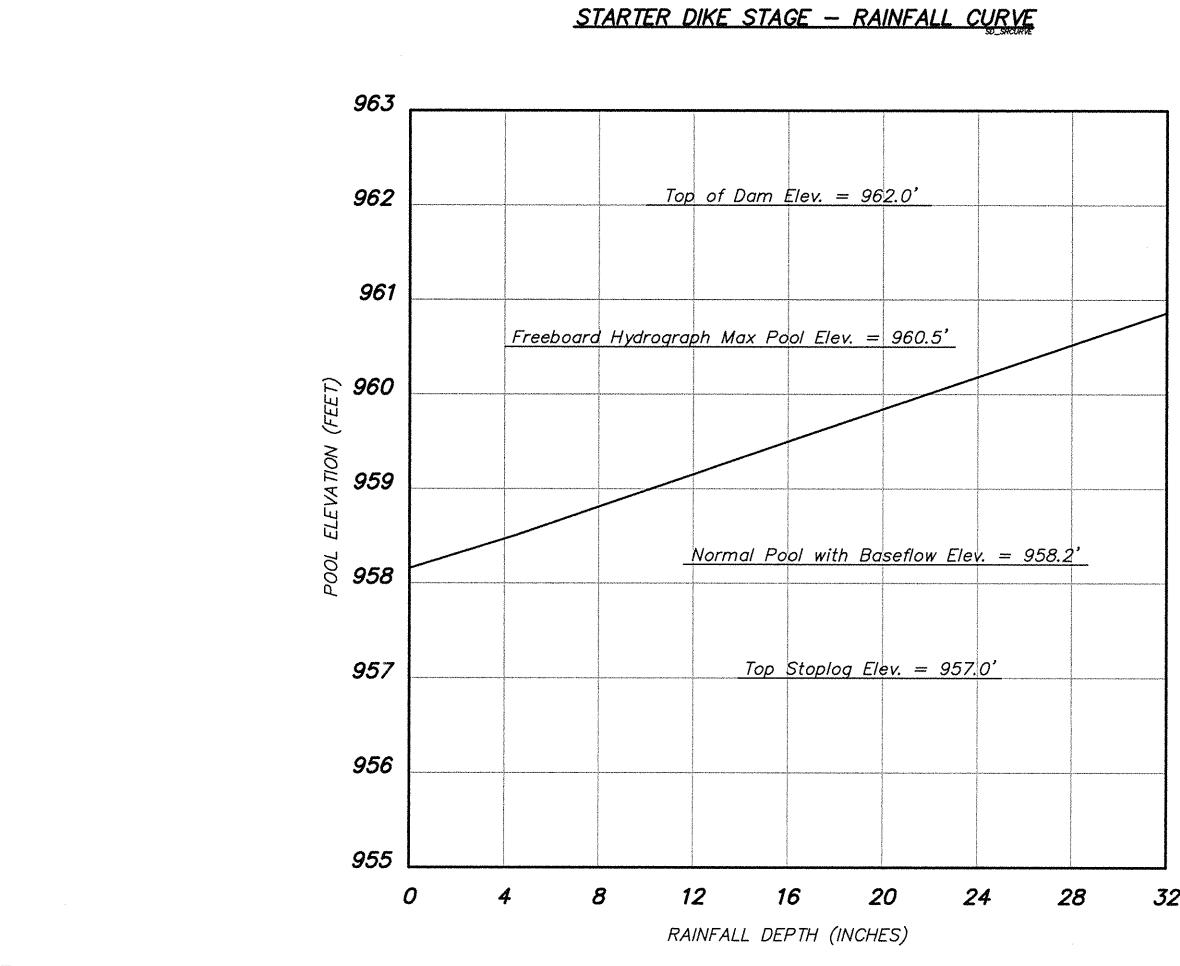
Capacity







FINAL CONFIGURATION PRINCIPAL SPILLWAY STAGE - DISCHARGE CURVE



## FINAL CONFIGURATION STAGE-RAINFALL CURVE

1. Rainfall Depth corresponds to an event having an SCS Type II storm distribution with a 6—hour duration.

Top of Dam Elev. 902.0'

Normal Pool with Baseflow Elev. 897.6'

Top Stoplog Elev. = 896.0'

RAINFALL DEPTH (INCHES)

Freeboard Hydrograph Max Pool Elev. 900.6'

902

901

F 900

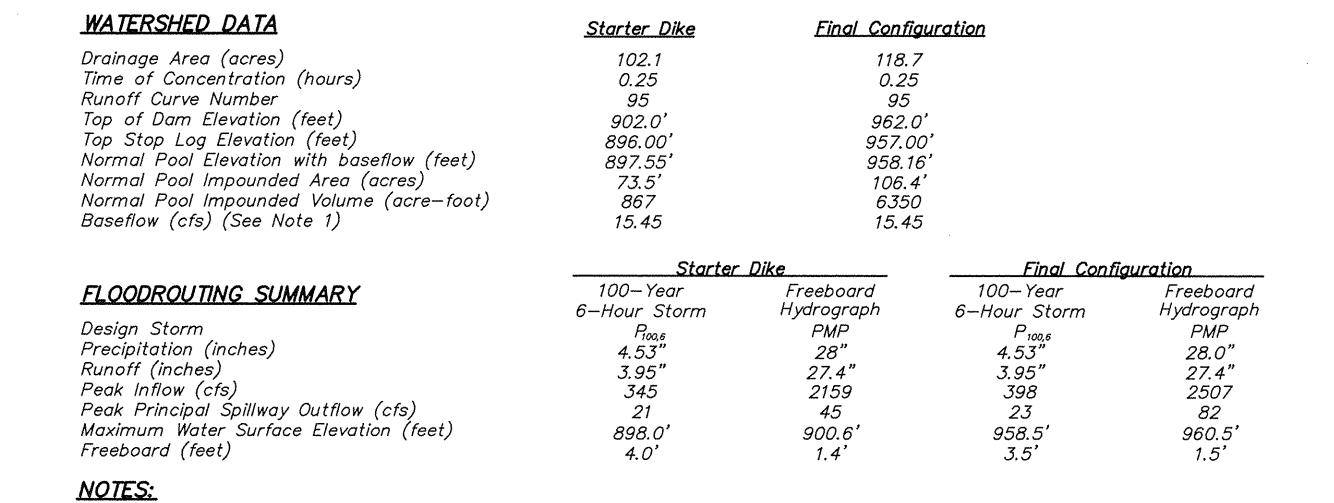
899

§ **898** 

897

896

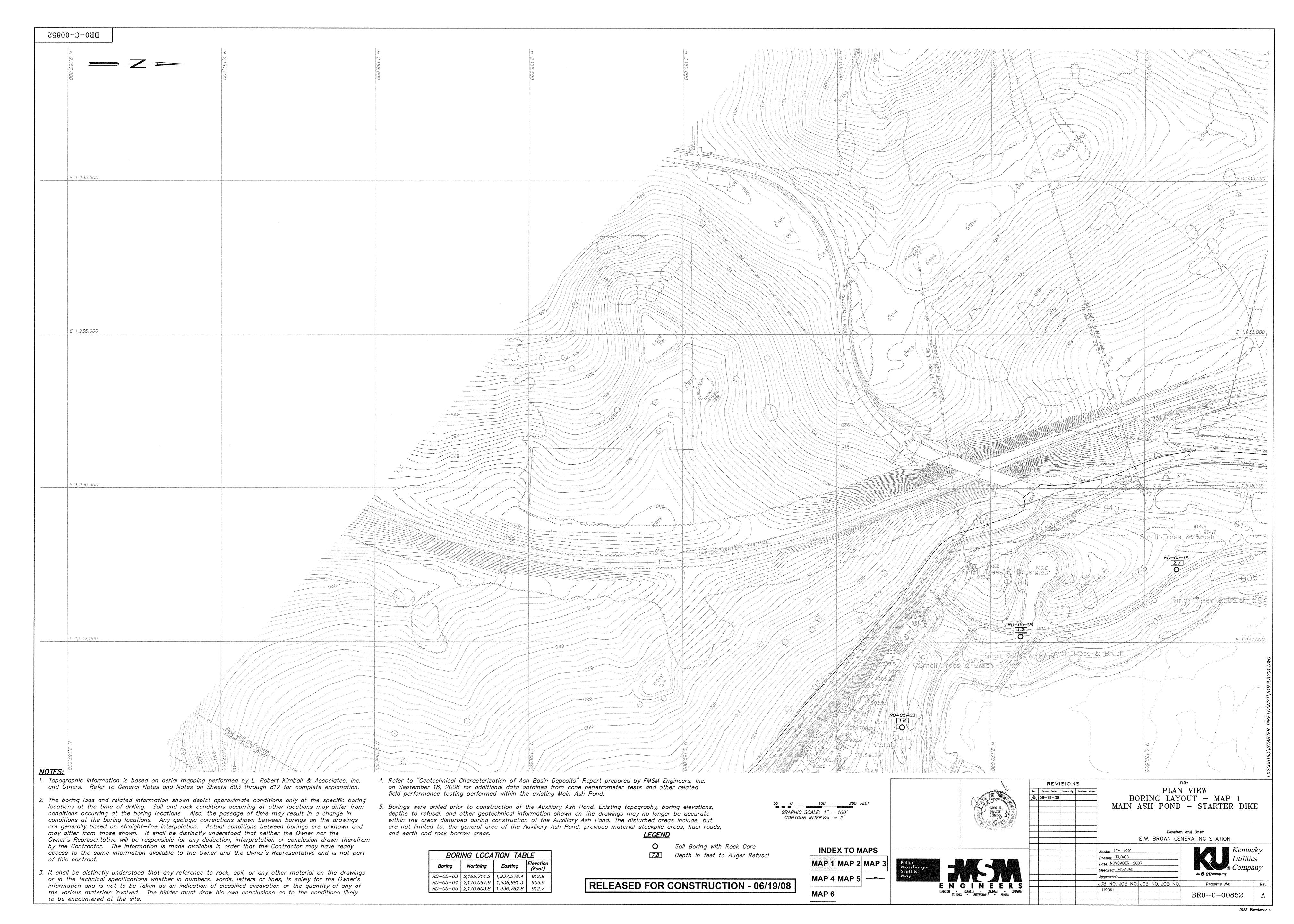
- 2. Pool Elevation indicates the peak stage for the constant process baseflow of 15.45 cfs plus the routing of the indicated 6—hour SCS Type II rainfall depth.

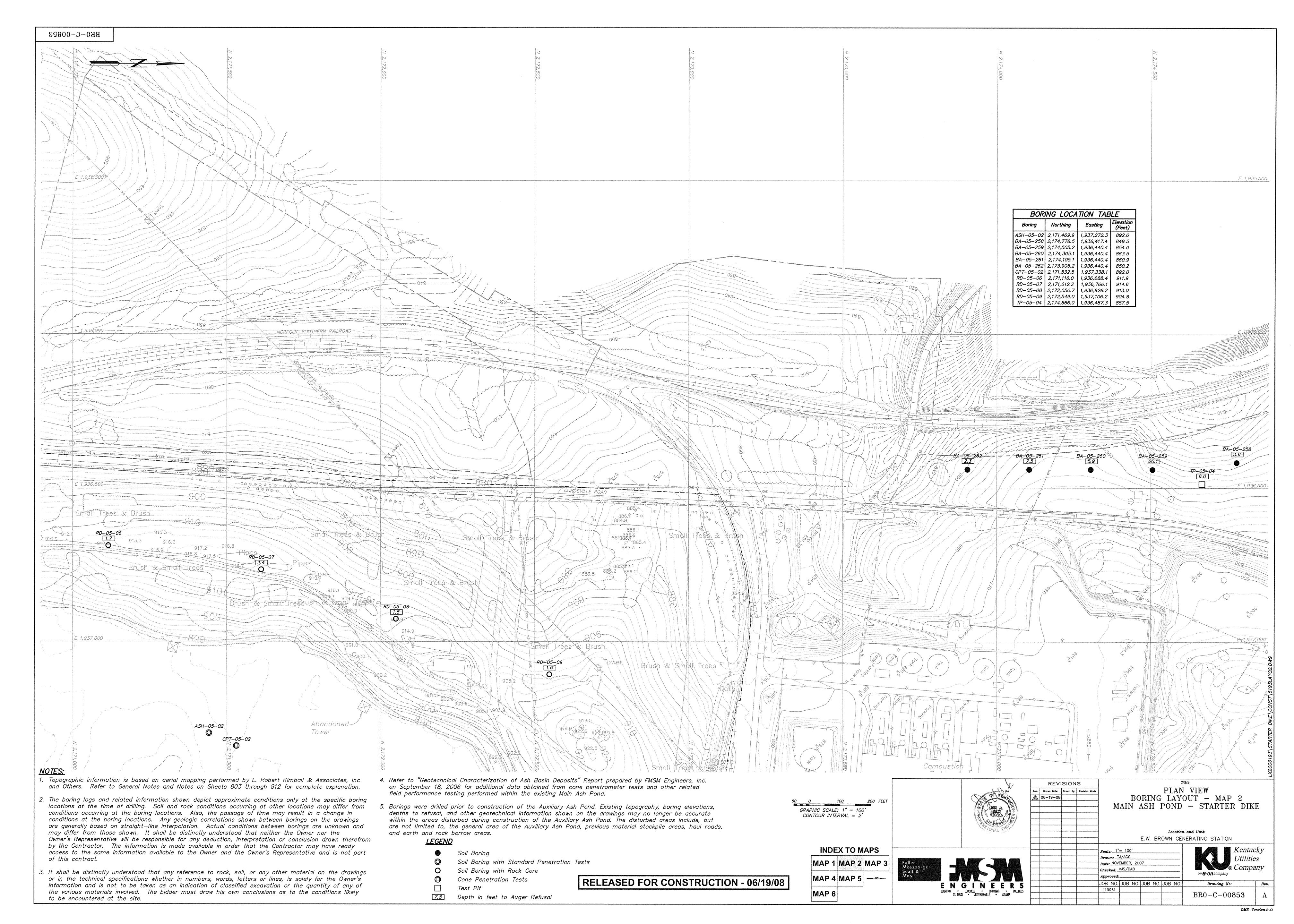


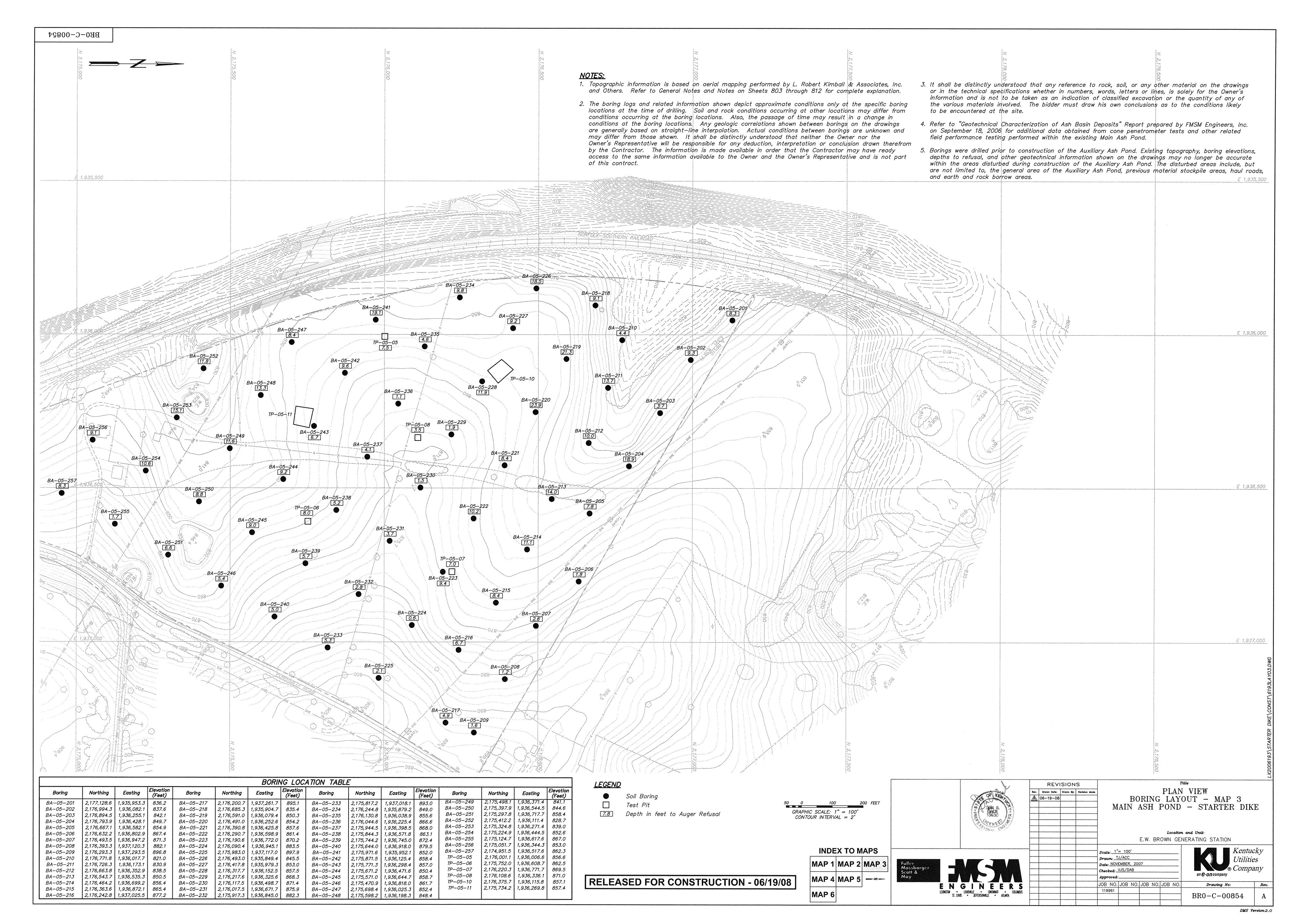
1. Process baseflow for the Main Ash Pond is 15.45 cfs which has been included in the storm hydrograph routing results.

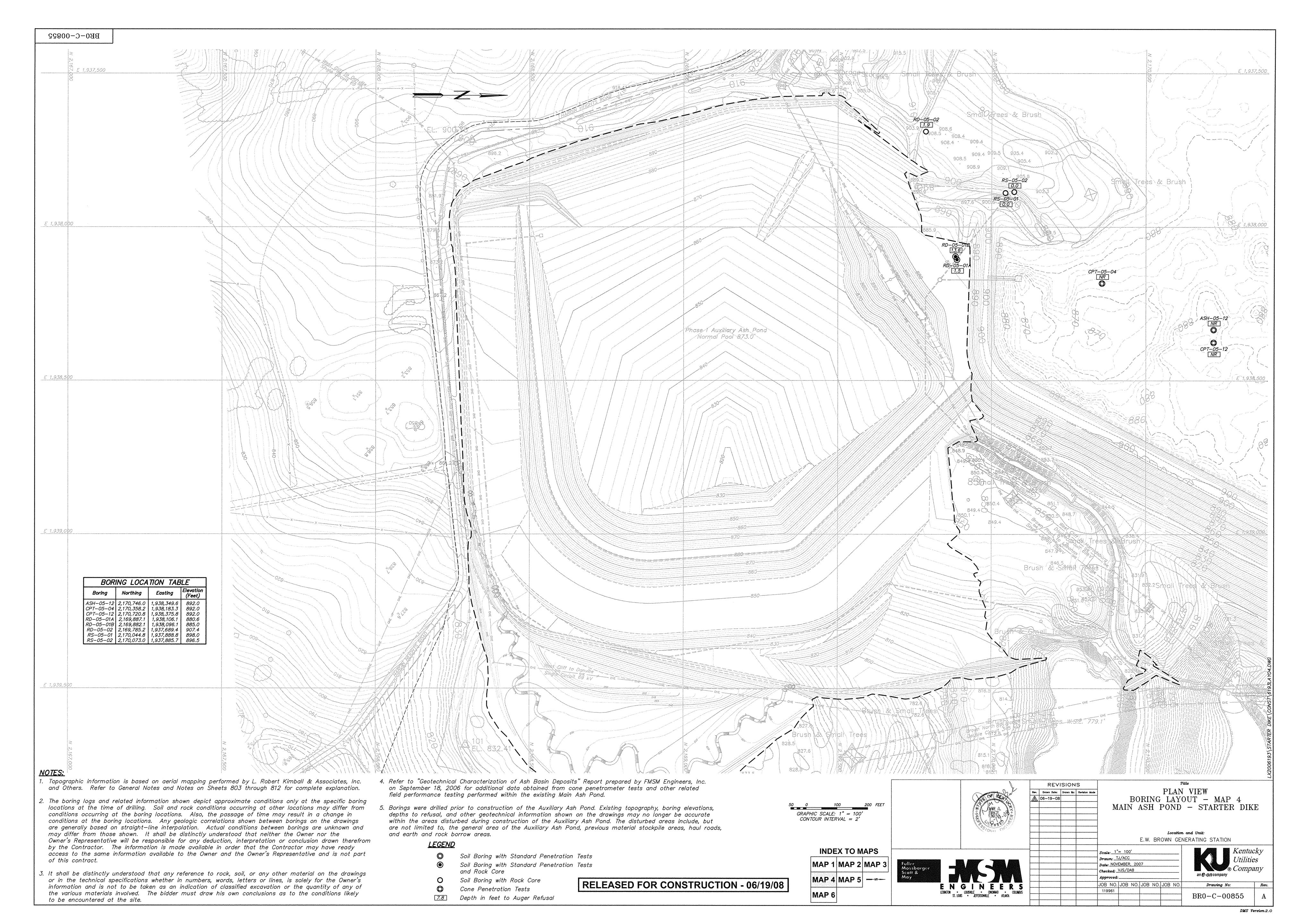
REVISIONS HYDRAULIC AND Rev. Drawn Date: Drawn By: Revision Made HYDROLOGIC DATA MAIN ASH POND - STARTER DIKE Location and Unit: E.W. BROWN GENERATING STATION *Kentucky* Scale: AS SHOWN Drawn: DMG Date: NOVEMBER, 2007 ® Company Checked: \_\_ an **e-on** company Approved:\_ JOB NO. JOB NO. JOB NO. JOB NO. ENGINERS
LECONSTON ST. LOUIS . JEFFERSONMALE . ATLANTA 119961 BR0-C-00851

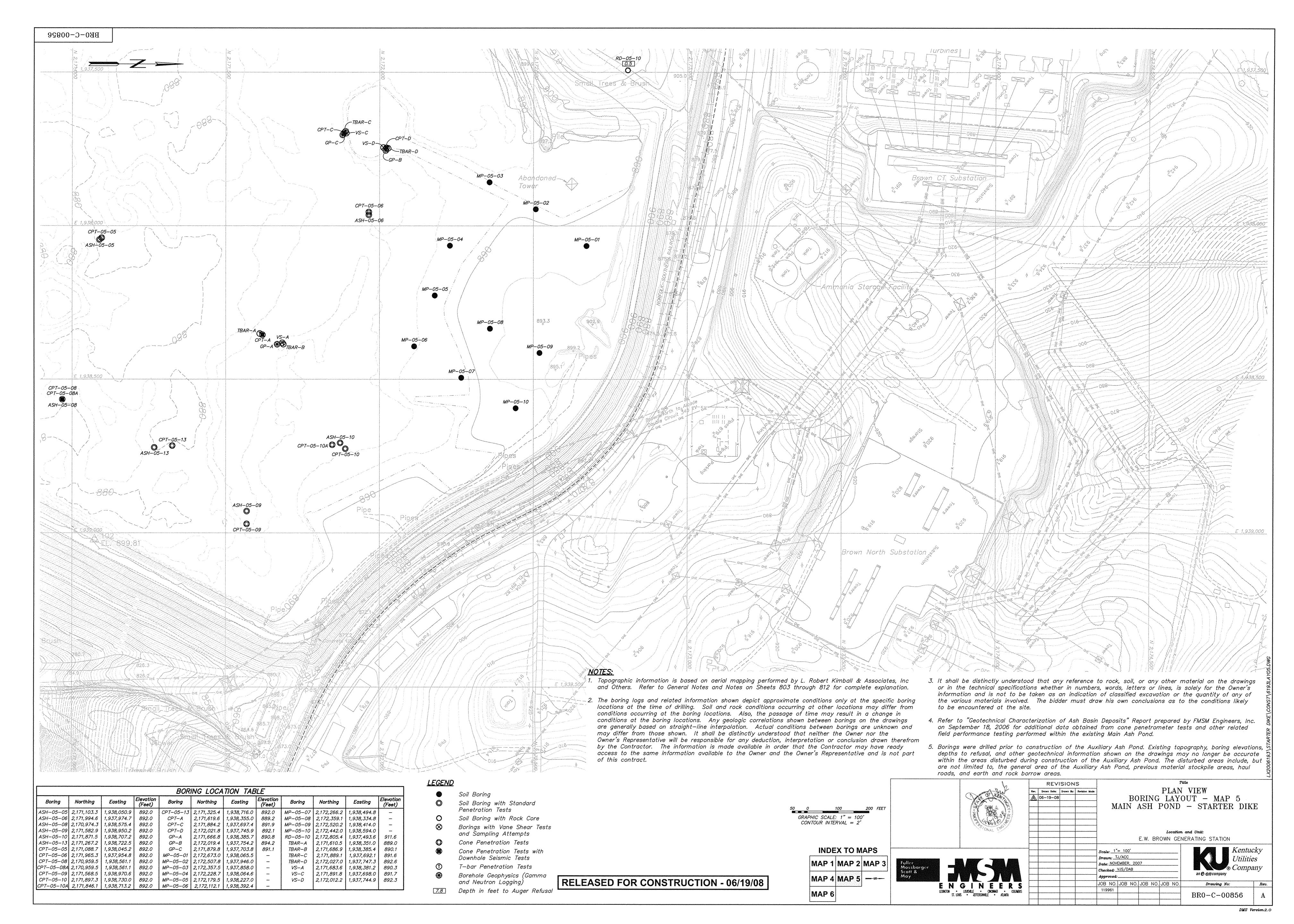
Drawing No:











1. Borings were drilled prior to construction of the Auxiliary Pond. Existing topography, boring elevations, depths to refusal and other geotechnical information shown on the drawings may no longer be accurate within the areas disturbed during construction of the Auxiliary Ash Pond. The disturbed areas include, but are not limited to, the general area of the Auxiliary Ash Pond, previous material stockpile areas, haul roads, and earth and rock borrow areas.

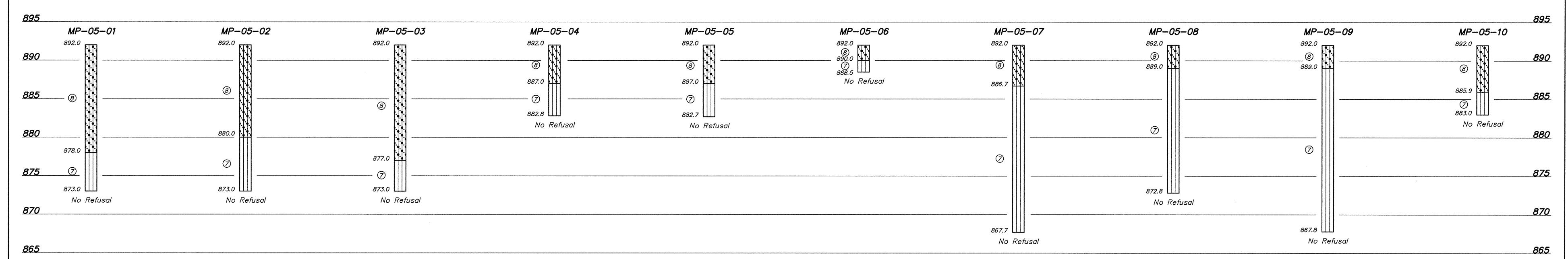
2. Refer to Sheet BRO-C-00864 for Legend, Soil Summary and Notes.

, & W		REV	ISION	S	Title
The state of the s	Rev.	Drawn Date:	Drawn By:	Revision Mode	LOGS OF BORINGS
The state of the s	A	06-19-08			MAIN ASH POND PERIMETER (RD)
and the second s					MAIN ASH POND - STARTER DIKÉ
· · · · · · · · · · · · · · · · · · ·					Location and Unit:
					E.W. BROWN GENERATING STATION
					Scale: AS SHOWN Kentuck
					Drawn: TJ/ACC Utilities
Fuller Mossbarger Scott &					Date: NOVEMBER, 2007 Checked: VJS/DAB  © Compan
Scott &					an <b>P.on</b> company
May					Approved:
ENGINEERS			<u> </u>		JOB NO. JOB NO. JOB NO. Drawing No:
EEXINGTON • LOUISVILLE • CINCINNATI • COLUMBUS St. Louis • Jeffersonville • Aitanta			<u> </u>		BR0-C-00857
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DMS Version 2.0

BB0-C-00828

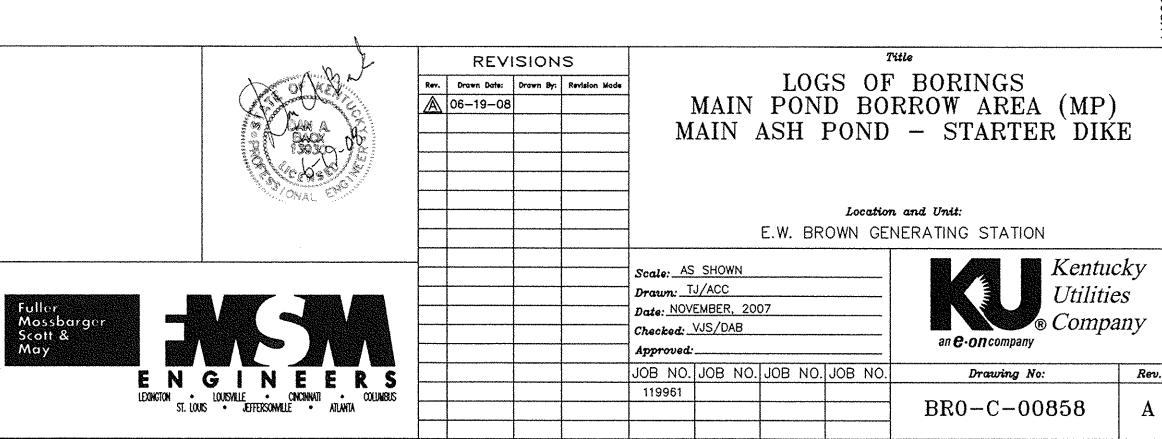
900



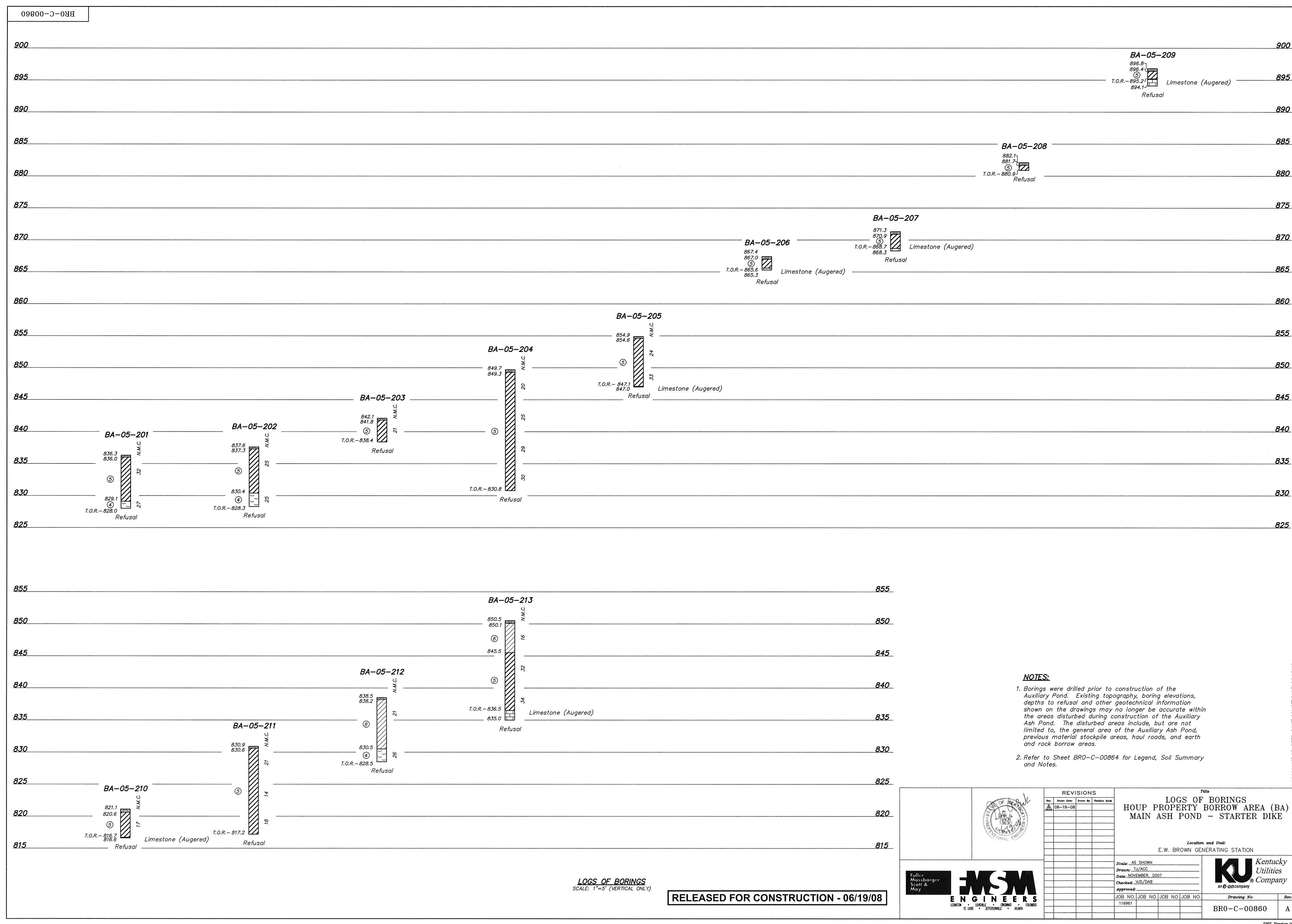
LOGS OF BORINGS SCALE: 1"=5' (VERTICAL ONLY)

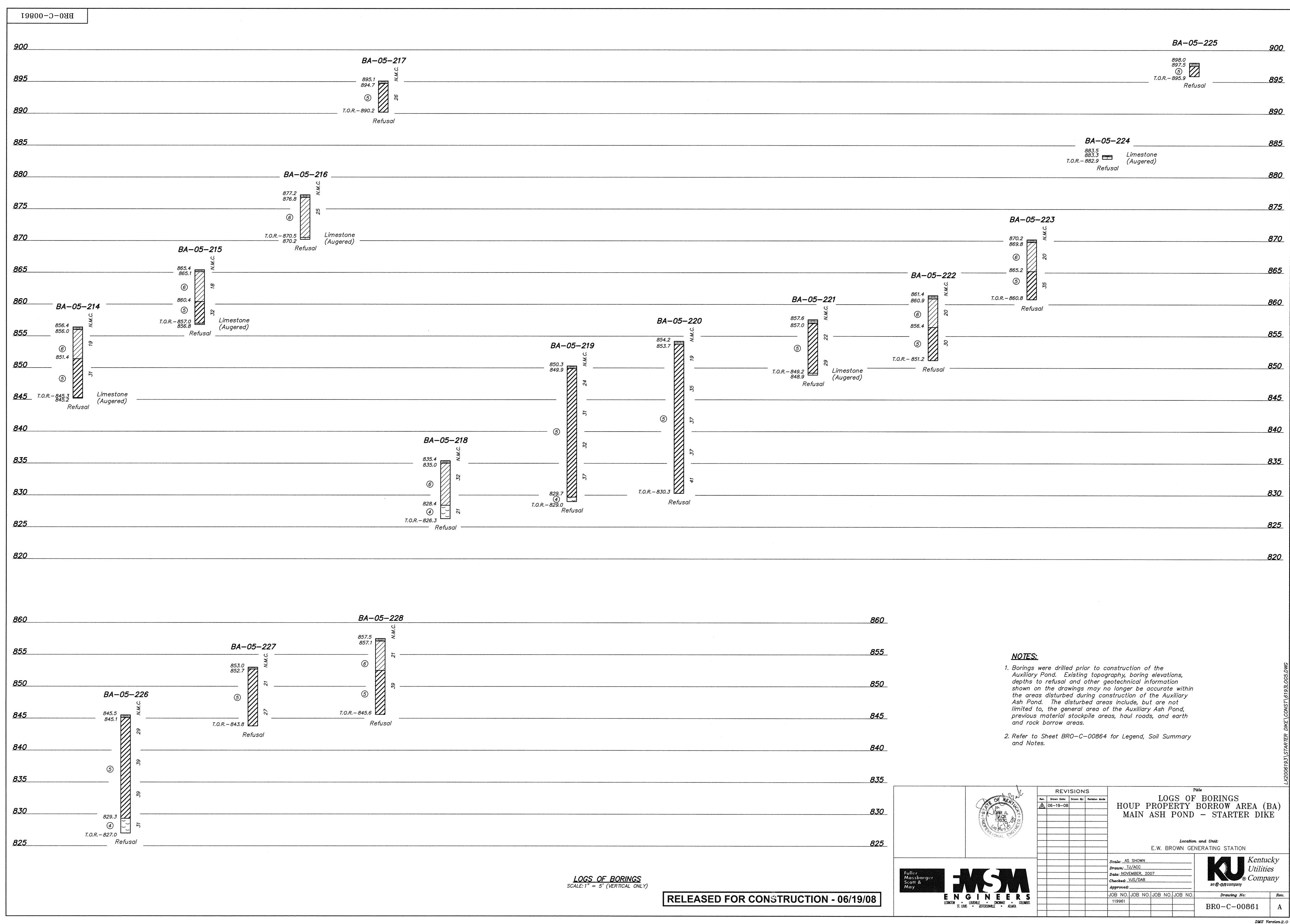
# NOTES:

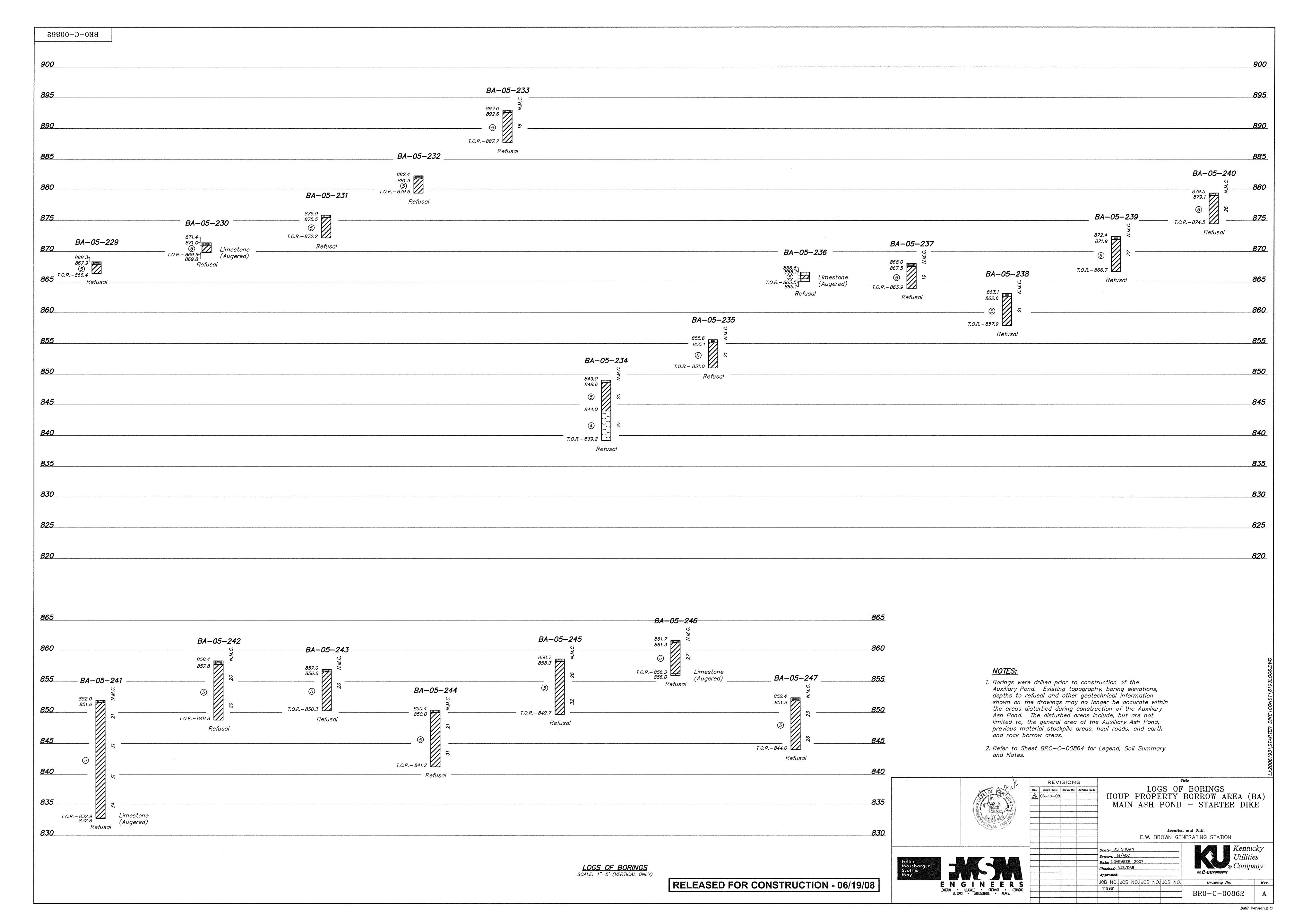
- 1. Borings were drilled prior to construction of the Auxiliary Pond. Existing topography, boring elevations, depths to refusal and other geotechnical information shown on the drawings may no longer be accurate within the areas disturbed during construction of the Auxiliary Ash Pond. The disturbed areas include, but are not limited to, the general area of the Auxiliary Ash Pond, previous material stockpile areas, haul roads, and earth and rock borrow areas.
- 2. Refer to Sheet BRO-C-00864 for Legend, Soil Summary and Notes.

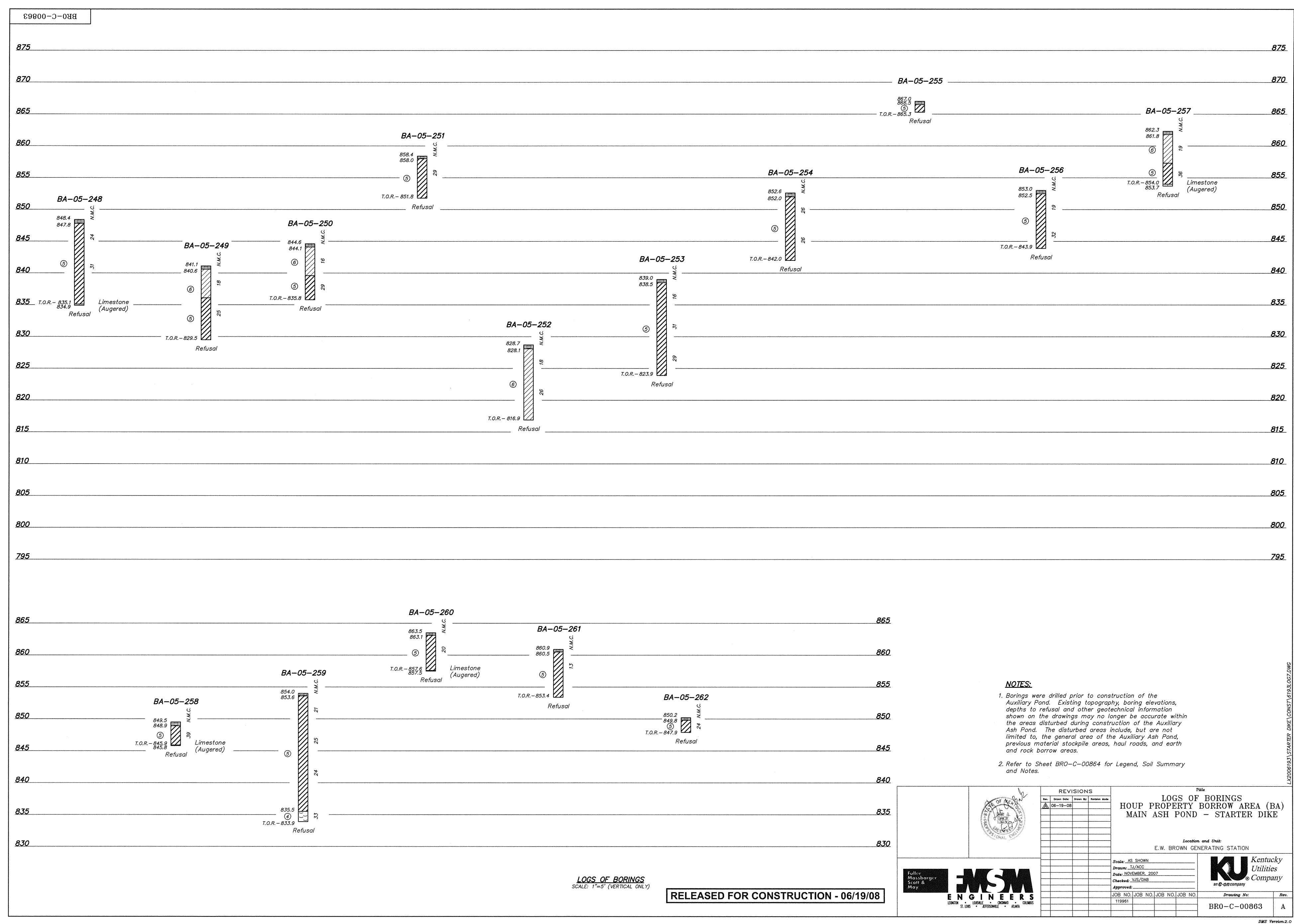












905		905
900	RS-05-01  RS-05-02  T.O.R. & B.C898.0	900
<u>895</u>	Limestone, gray, fine to  coarse, crystalline grained, hard, thin bedded, irregular/  Limestone, gray, fine to  Limestone, gray, fine to	895
<u>890</u>	nodular bedding, fossiliferous  with shale stringers and partings.  888.0  nodular bedding, fossiliferous  with shale stringers and partings.  coarse, crystaime grainea, hard, thin bedded, irregular/ nodular bedding, fossiliferous with shale stringers and partings.	<i>890</i>
<i>885</i>	886.5	885
880		880
<i>875</i>		875

LOGS OF BORINGS SCALE: 1"=5' (VERTICAL ONLY)

NOTES:

1. Borings were drilled prior to construction of the Auxiliary Pond. Existing topography, boring elevations, depths to refusal and other geotechnical information shown on the drawings may no longer be accurate within the areas disturbed during construction of the Auxiliary Ash Pond. The disturbed areas include, but are not limited to, the general area of the Auxiliary Ash Pond, previous material stockpile areas, haul roads, and earth and rock borrow areas.

2. Refer to Sheet BRO-C-00864 for Legend, Soil Summary and Notes.

|--|

Topsoil/Rootzone

Limestone layers, cobbles, gray, highly weathered

Lean to Fat Clay, medium brown, moist, medium stiff, with manganese

and iron concretions (Field Classified)

Fat Clay, medium brown, moist to wet, soft to medium stiff.

Lean to Fat Clay, light brown, moist, medium stiff, with manganese concretions and chert fragments. (Field Classified)

Clay Shale (Bentonite), greenish—gray, laminated, and soft. (Field Classified)

Lean Clay, medium brown, moist to wet, soft to medium stiff. (Houp Property)

Fat Clay, medium brown, moist to wet, soft to medium stiff. (Houp Property)

Silt, gray, non-plastic, wet, loose. (Fly Ash, Hydraulically Placed)

Silty coarse sand with gravel, gray and black, non-plastic, wet, loose. (Bottom Ash with Variable Fines, Hydraulically Placed)

Undisturbed Thin-Walled (Shelby) Tube Sample

Standard Penetration Test Interval Standard Penetration Test Blow Count (blows/ft.)

Natural Moisture Content (%) U. W. W. Unit Weight Wet (lbs./cu.ft.)

Unit Weight Dry (lbs./cu.ft.)

Unconfined Compressive Strength (tons/sq. ft.) T.O.R. - Top of Rock (Indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of

weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.) B.C.- Begin Rock Core

E.O.C. – End Rock Core

Refusal Auger Refusal using a carbide—tipped tooth auger bit

No Refusal No Refusal Encountered R.Q.D. Rock Quality Designation(%)

Recovery(%) REC.

Weight of Rods (Penetration without Hammer Blow) Blows/6-in. Penetration Test Blow Count (blows/6-in.)

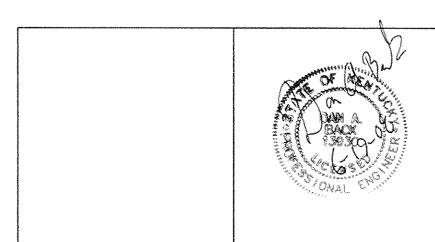
OFF SITE MATERIALS SUMMARY SAMPLE DESCRIPTION

Composition of Total Sample SLIT(-No.200 +0.005mm)

CLAY(-0.005mm) Non Plastic | Non Plastic | Non Plastic | Non Plastic 2.36 2.61 2.74 2.27 A-4(0) A-4(0) A-1-b(0) A-4(0) AASHTO CLASSIFICATION UNIFIED CLASSIFICATION CALIF. BEARING RATIO MAXIMUM DRY DENSITY (pcf) OPTIMUM MOISTURE (%) 17.6
PERMEABILITY (CM/SEC) 9.12E-05
ANGLE OF INTERNAL FRICTION (°) 43.1 EFFECTIVE COHESION (PSF)

SOIL SUMMARY												
SAMPLE NO.	5	6	7	7	7	7	7	7	7	7	8	8
STATION	BA-05-244	TP-05-11	ASH-05-02	ASH-05-05	ASH-05-06	ASH-05-08	ASH-05-08	ASH-05-12	ASH-05-13	ASH-05-13	ASH-05-09	ASH-05-10
OFFSET		_	-		_	-	-	_	_		<del>-</del>	
DEPTH	0.4'-9.2'	8.0'-9.0'	15.0'-33.0'	20.0'-45.5'	15.0'-39.0'	40.0'-68.0'	69.0'-93.5'	40.0'-56.0'	50.0'-71.0'	90.0'-108.0'	10.0'-25.0'	15.0'-31.0'
GRAVEL(-3" + No.4)		0	0	0	1	13	0	0	1	13	<i>38</i>	19
Composition GRAVEL(-3" +No.4) SAND(-No.4 +No.200) SILTY No.200 +0.005 mm	_	14	7	20	9	28	10	6	<i>25</i>	31	51	50
Sample Dilli No.200 To.000 IIII	1) -	38	86	71	79	51	82	75	67	54	10	28
CLAY(-0.005mm)		48	7	9	11	8	8	19	7	2	1	3
LIQUID LIMIT	59	42		-	-		-			_	•	_
PLASTIC LIMIT	23	21	Non Plastic	Non Plastic	Non Plastic							
PLASTICITY INDEX	36	21	~	-	_			_				_
ACTIVITY INDEX		0.53	-		***	_			_		<del>-</del>	
SPECIFIC GRAVITY	<b>–</b>	2.79	2.19	2.40	2.28	2.29	2.20	2.30	2.20	2.64	2.42	2.17
AASHTO CLASSIFICATION	-	A-7-6(19)	A-4(0)	A-1-b(0)	A-2-4(0)							
UNIFIED CLASSIFICATION	CH	CL	ML	SP-SM	SM							
CALIF. BEARING RATIO		_			_			-		<del>-</del>	*****	-
MAXIMUM DRY DENSITY (pcf)	94.2	99.4				••••	-	-	***	-		_
OPTIMUM MOISTURE (%)	26.5	23.8	-				_	****	_			

The boring logs and related information shown depict approximate conditions only at the specific boring locations at the time of drilling. Soil and rock conditions occurring at other locations may differ from conditions occurring at the boring locations. Also, the passage of time may result in a change in conditions at the boring locations. Any geologic correlations shown between borings on the drawings are generally based on straight—line interpolation. Actual conditions between borings are unknown and may differ from those shown. It shall be distinctly understood that neither the Owner nor the Owner's Representative will be responsible for any deduction, interpretation or conclusion drawn therefrom by the Contractor. The information is made available in order that the Contractor may have ready access to the same information available to the Owner and the Owner's Representative and is not part of this contract.



REVISIONS LOGS OF BORINGS Rev. Drawn Date: Drawn By: Revision Mode ▲ 06-19-08 RISER STRUCTURE (RS) MAIN ASH POND - STARŤEŔ DIKE

> Location and Unit: E.W. BROWN GENERATING STATION

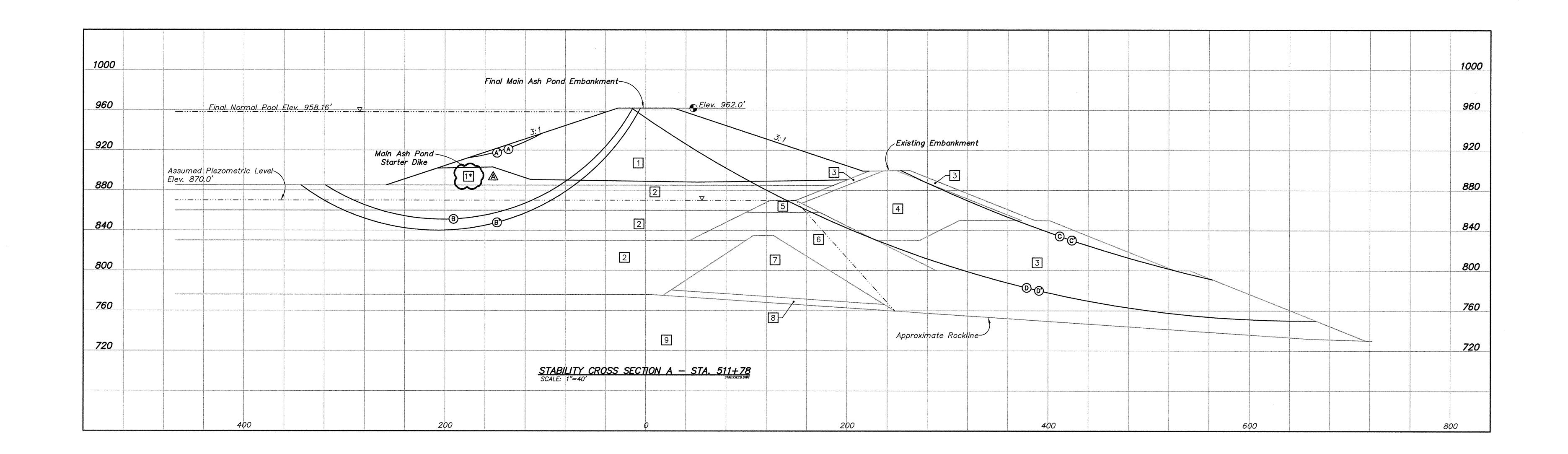
ENGINE ERS
LEDINGTON ST. LOUIS . LETTERSONMILE . ATLANTA

Scale: AS SHOWN Drawn: TJ/ACC Date: NOVEMBER, 2007 Checked: VJS/DAB Approved: \_\_\_ JOB NO. JOB NO. JOB NO. JOB NO.

Kentucky Utilities an **e-on** company Drawing No:

RELEASED FOR CONSTRUCTION - 06/19/08

DMS Version 2.0

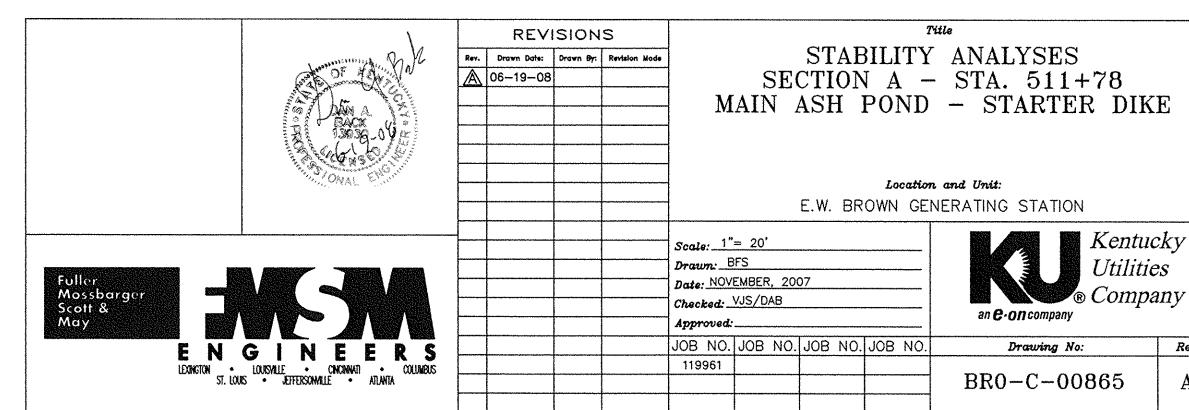


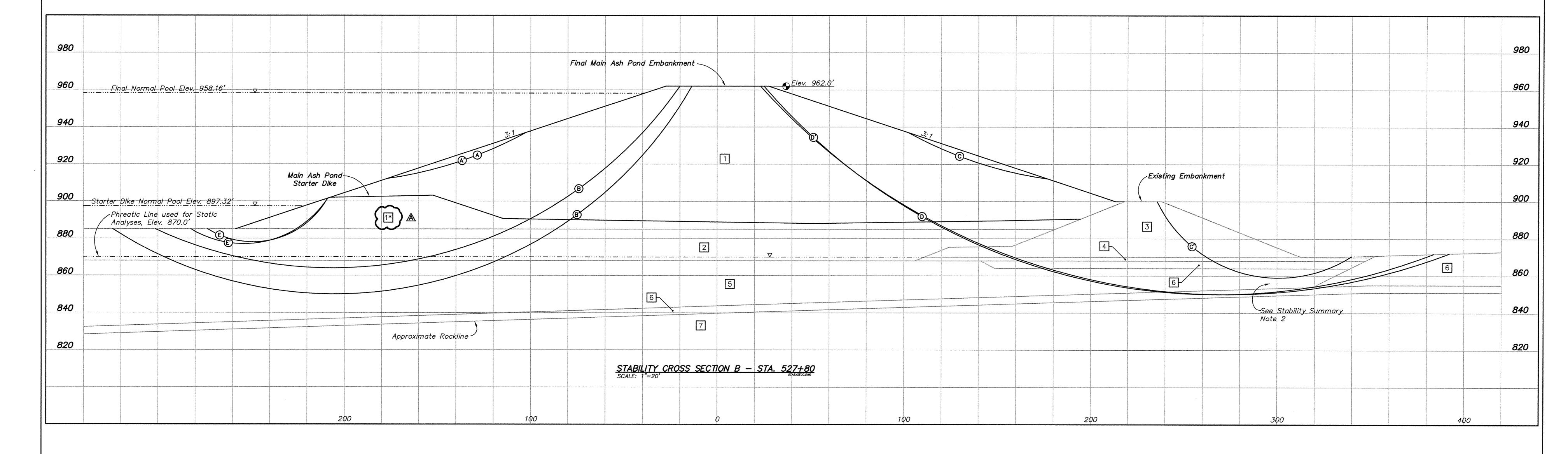
Failure Surface	Failure Condition	Failure Mode	Factor of Safety		
Α	Long Term, Upstream		Static	2.1	
A'	Shallow Failure – No Pool	Rotational	Dynamic <sup>(1)</sup>	1.8	
В	Long Term, Upstream Deep Failure — No Pool	Detetional	Static	2.4	
B'		Rotational	Dynamic <sup>(1)</sup>	1.7	
С	Long Term, Downstream	Rotational	Static	2.0	
C'	Shallow Failure		Dynamic <sup>(1)</sup>	1.7	
D	Long Term, Downstream	0-1-1:	Static	<i>2.5</i>	
D'	Deep Failure	Rotational	Dynamic <sup>(1)</sup>	2.1	

<sup>(1)</sup> The factors of safety under dynamic (pseudo—static) loading conditions are based on a peak ground acceleration (k<sub>max</sub>) value of 0.100g for a 2 percent probability of exceedance in 50 years.

		SUMMARY OF SHEAR ST	RENGTH F	PARAMETER	S			
	Material	Description	Effective Stress					
	No.	Description	¯ (p.s.f.)	$\overline{\phi}^{\bullet}$ (deg.)	γ (p.c.f.)			
		Gypsum	0	35	110			
$\mathbb{A}$	1*	Type IIa-24	Ö	<i>35</i>	110			
	2	Fly Ash	0	32	100			
	3	Rock Fill	0	<i>38</i>	115			
İ	4	1990 Embankment	0	<i>32</i>	118			
	5	1970's Embankment	0	28	125			
	6	1970's Embankment	0	30	125			
	7	Original Embankment	0	<i>32</i>	125			
	8	Foundation Soil	0	<i>32</i>	125			
	9	Bedrock	0	Very Strong	125			

NOTE: Refer to Design Report for discussion of material zones and material strength parameters. Type IIa—24 Rock Embankment material parameters are assumed to be equal to gypsum parameters due to 20 percent maximum allowable fine content and possible shale material present in the Type IIa—24 rock (cohesion is neglected).



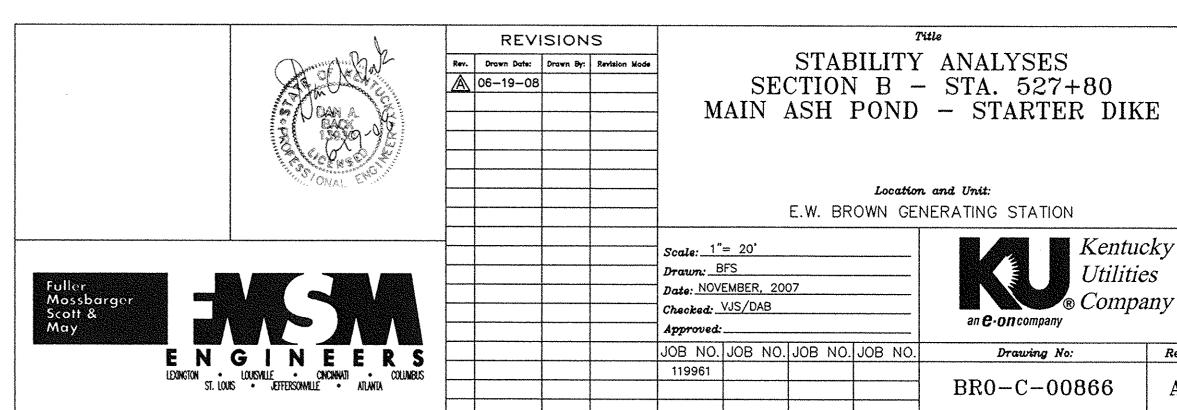


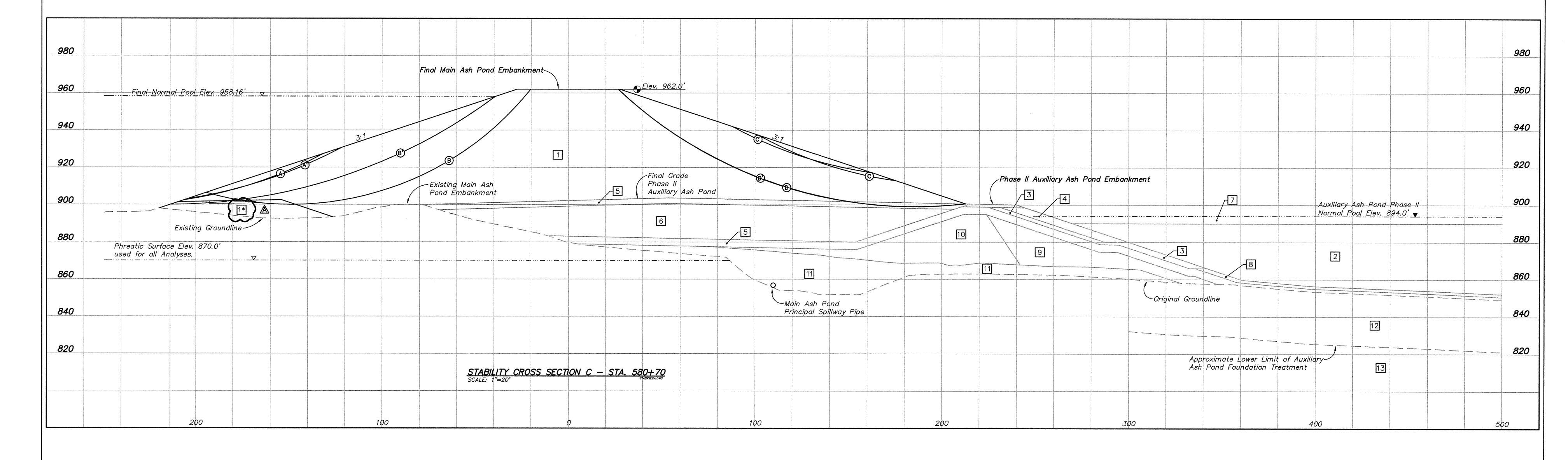
SUMMARY OF SLOPE STABILITY ANALYSES						
Failure Surface	Failure Condition	Failure Mode	Factor of Safety			
Α	Long Term, Upstream	Dotational	Static	2.1		
A'	Shallow Failure - No Pool	Rotational	Dynamic $\binom{(1)}{(2)}$	1.8		
В	Long Term, Upstream Deep Failure — No Pool	Rotational	Static	2.3		
B'			Dynamic $\binom{(1)}{(2)}$	1.8		
С	Long Term, Downstream Shallow Failure	Rotational	Static	2.1		
C'		Kotational	Dynamic $\binom{1}{2}$	1.3		
D	Long Term, Downstream	0-4-41	Static	2.2		
D'	Deep Failure	Rotational	Dynamic $\binom{(1)}{(2)}$	1.6		
Ε	Long Term	Rotational	Static	2.0		
E'	Starter Dike — No Pool	Kotational	Dynamic <sup>(1)</sup>	1.4		
(1) T			\	44.1.4		

(1) The factors of safety under dynamic (pseudo-static) loading conditions are based on a peak ground acceleration (k<sub>max</sub>) value of 0.100g for a 2 percent probability of exceedance in 50 years.
(2) Ultimate Long Term Phreatic Surface must be below Elev. 856' to avoid potential liquefaction of the underlying fly ash.

	SU	MMARY OF SHEAR	STRENGTH	H PARAMET	ERS		
	Material	Docorintian	Effective Stress				
	No.	Description	¯ (p.s.f.)	$\bar{\phi}^{m{\cdot}}$ (deg.)	γ (p.c.f.)		
	1	Gypsum	0	35	110		
$\mathbb{A}$		Type IIa-24	0	35	110		
	2	Fly Ash	0	32	100		
	3	Existing Embankment	0	<i>32</i>	118		
	4	Old Working Platform	0	<i>35</i>	115		
	5	Bottom Ash	0	<i>32</i>	115		
	6	Foundation Soil	0	32	100		
	7	Bedrock	0	Very Strong	125		

NOTE: Refer to Design Report for discussion of material zones and material strength parameters. Type IIa—24 Rock Embankment
material parameters are assumed to be equal to gypsum parameters
due to 20 percent maximum allowable fine content and possible shale material present in the Type IIa—24 rock (cohesion is neglected).



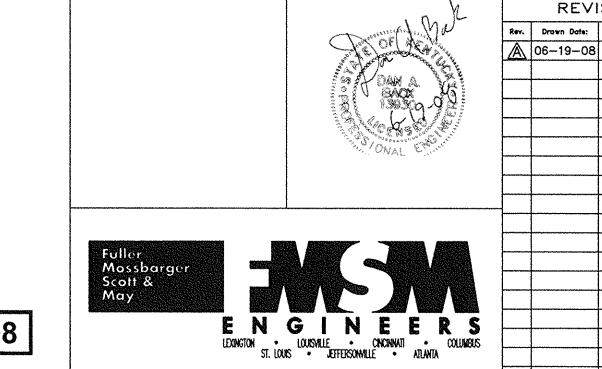


SUMMARY OF SLOPE STABILITY ANALYSES						
Failure Surface	Failure Condition	Failure Mode	Factor of Safety			
Α	Long Term, Upstream		Static	2.1		
A'	Shallow Failure – No Pool	Rotational	Dynamic <sup>(1)</sup>	1.8		
В	Long Term, Upstream Deep Failure — No Pool	Rotational	Static	2.5		
B'			Dynamic <sup>(1)</sup>	1.9		
С	Long Term, Downstream	D-4-1:	Static	2.1		
C'	Shallow Failure	Rotational	Dynamic <sup>(1)</sup>	1.8		
D	Long Term, Downstream	Potation -	Static	2.2		
D'	Deep Failure	Rotational	Dynamic <sup>(1)</sup>	1.9		

<sup>(1)</sup> The factors of safety under dynamic (pseudo—static) loading conditions are based on a peak ground acceleration (k<sub>max</sub>) value of 0.100g for a 2 percent probability of exceedance in 50 years.

SU	MMARY OF SHEAR	STRENGT	H PARAMET	TERS			
Material	Description	Effective Stress					
No.	Description	¯ (p.s.f.)	$\overline{\phi}^{ullet}(deg.)$	γ (p.c.f.)			
	Gypsum	0	35	110			
	Type IIa-24	Ö	35	110			
2	Fly Ash	0	32	100			
3	No. 57 Stone	0	<i>38</i>	110			
4	Rock Fill	0	<i>38</i>	118			
5	Clay	100	28	118			
6	Gypsum	0	<i>35</i>	118			
7	Water	0	0	62.4			
8	Bottom Ash	0	<i>38</i>	118			
9	Clay/Rock Fill	100	28	118			
10	Rock/Clay Fill	100	28	118			
11	Rock Fill	0	<i>38</i>	118			
12	Blasted Zone	0	28	118			
13	Bedrock	0	Very Strong	125			

NOTE: Refer to Design Report for discussion of material zones and material strength parameters. Type IIa—24 Rock Embankment material parameters are assumed to be equal to gypsum parameters due to 20 percent maximum allowable fine content and possible shale material present in the Type IIa—24 rock (cohesion is neglected). 



REVISIONS STABILITY ANALYSES
SECTION C - STA. 580+70
MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Made

> Location and Unit: E.W. BROWN GENERATING STATION

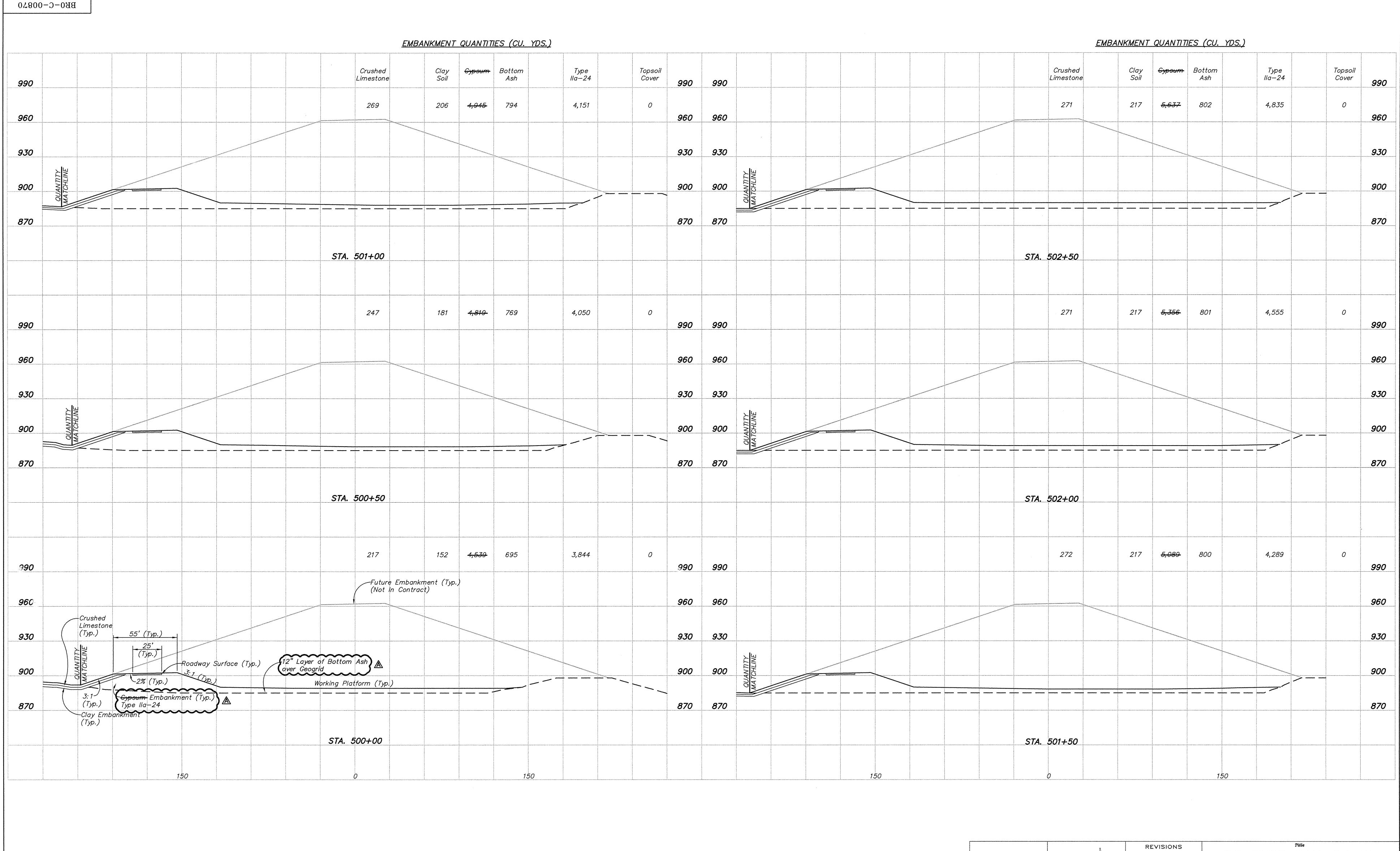
Scale: 1"= 20' Drawn: BFS Date: NOVEMBER, 2007 an **e·on** company

Approved:\_\_\_\_

Checked: VJS/DAB JOB NO. JOB NO. JOB NO. JOB NO. BR0-C-00867

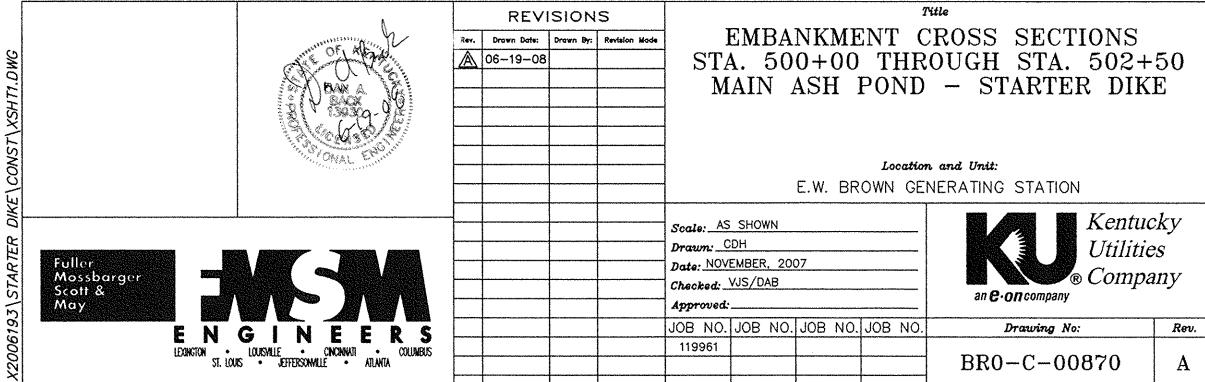
RELEASED FOR CONSTRUCTION - 06/19/08

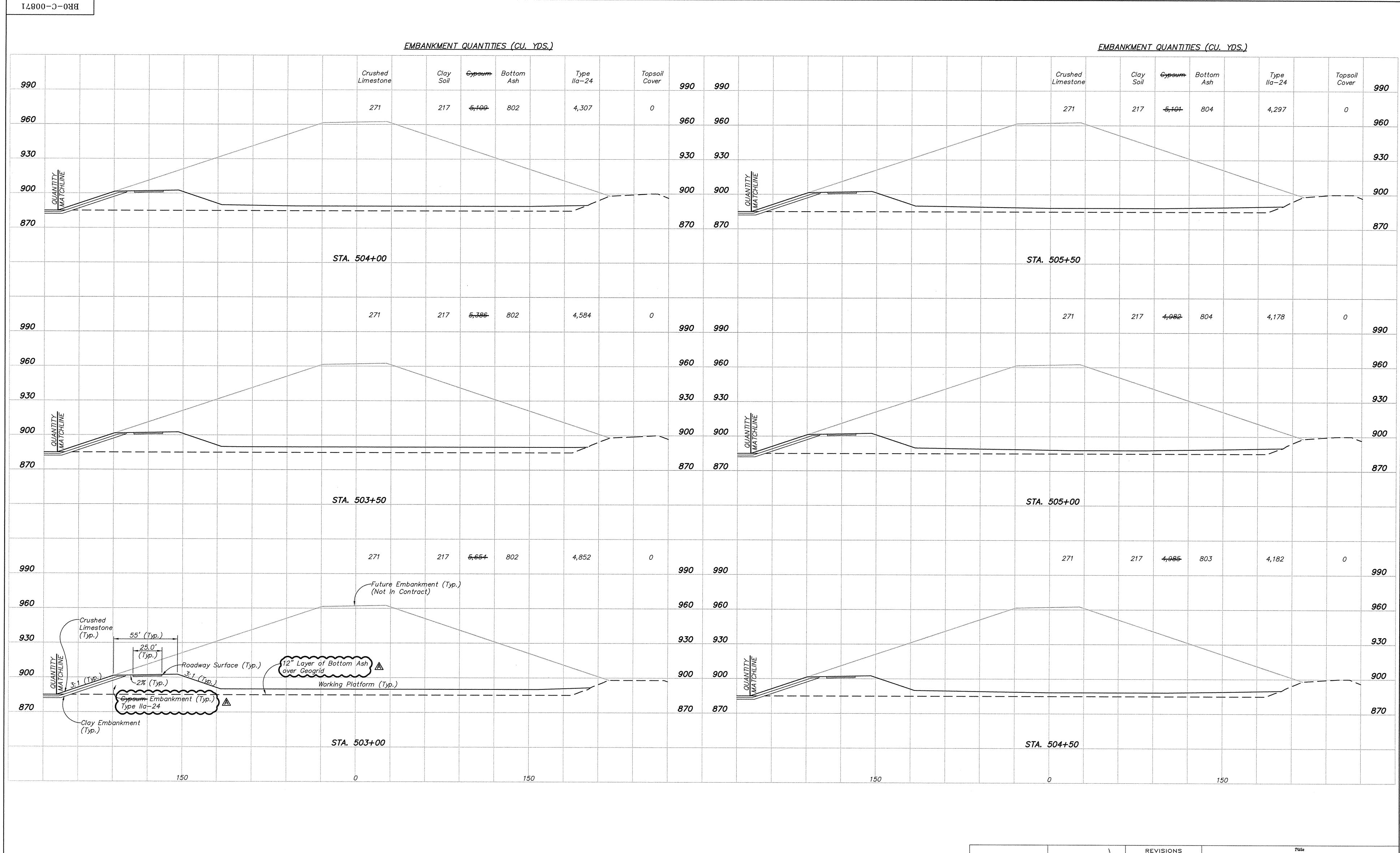
Drawing No:



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.





1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

Foller Mossbarger Scott & May

ENGINE NEERS COUNTIL COUNTILS COUNT

REVISIONS

Rev. Drawn Date: Drawn By: Revision Mode

A 06-19-08

STA. 503+00 THROUGH STA. 505+50

MAIN ASH POND - STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN

Drawn: CDH

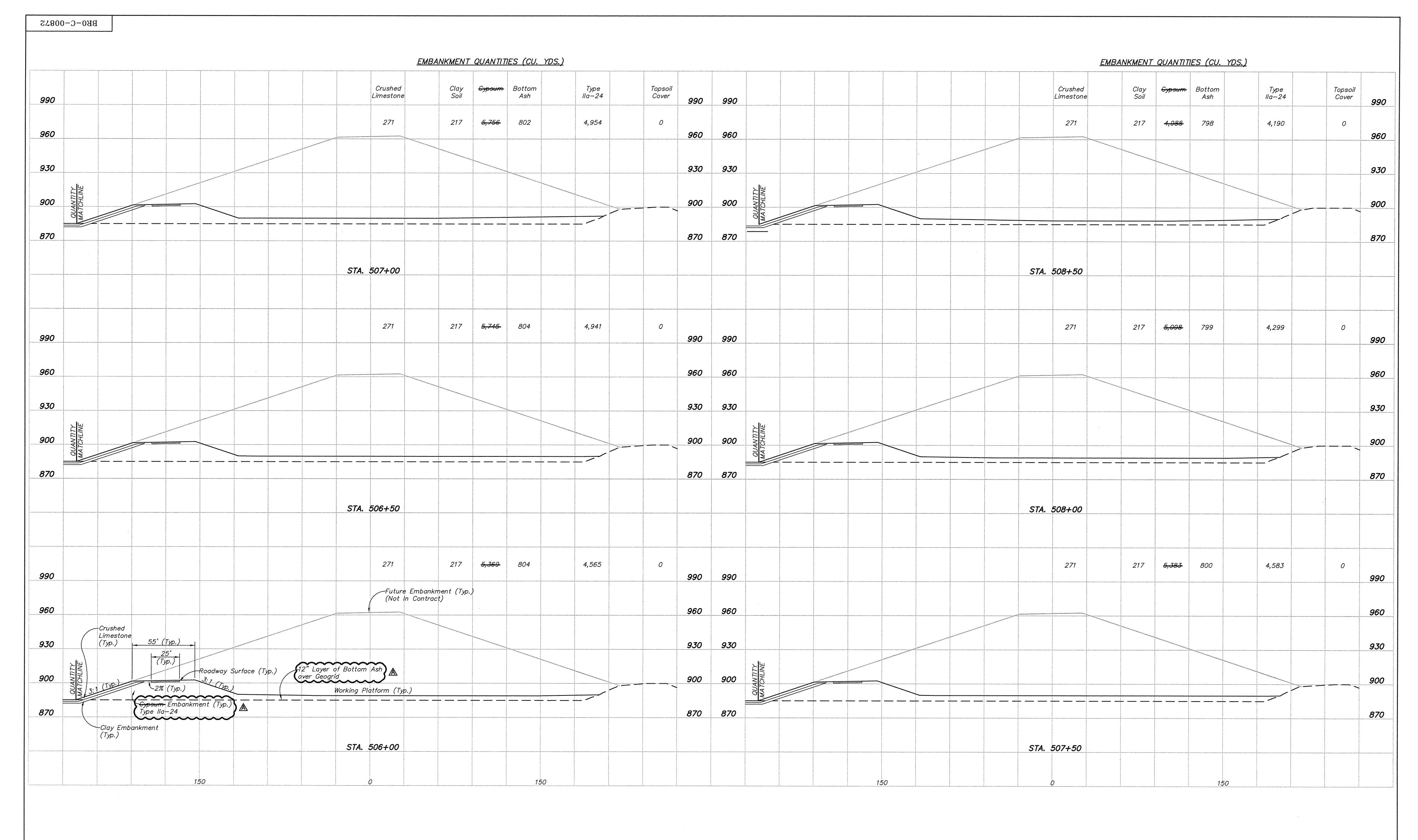
Date: NOVEMBER, 2007

Checked: VJS/DAB

Approved:

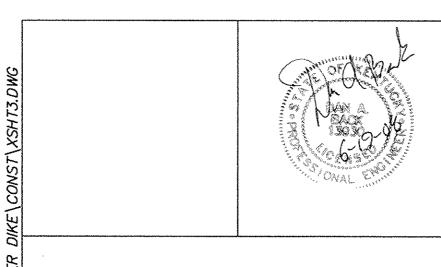
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RELEASED FOR CONSTRUCTION - 06/19/08



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



REVISIONS EMBANKMENT CROSS SECTIONS STA. 506+00 THROUGH STA. 508+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Made <u>A</u> 06-19-08

> Location and Unit: E.W. BROWN GENERATING STATION

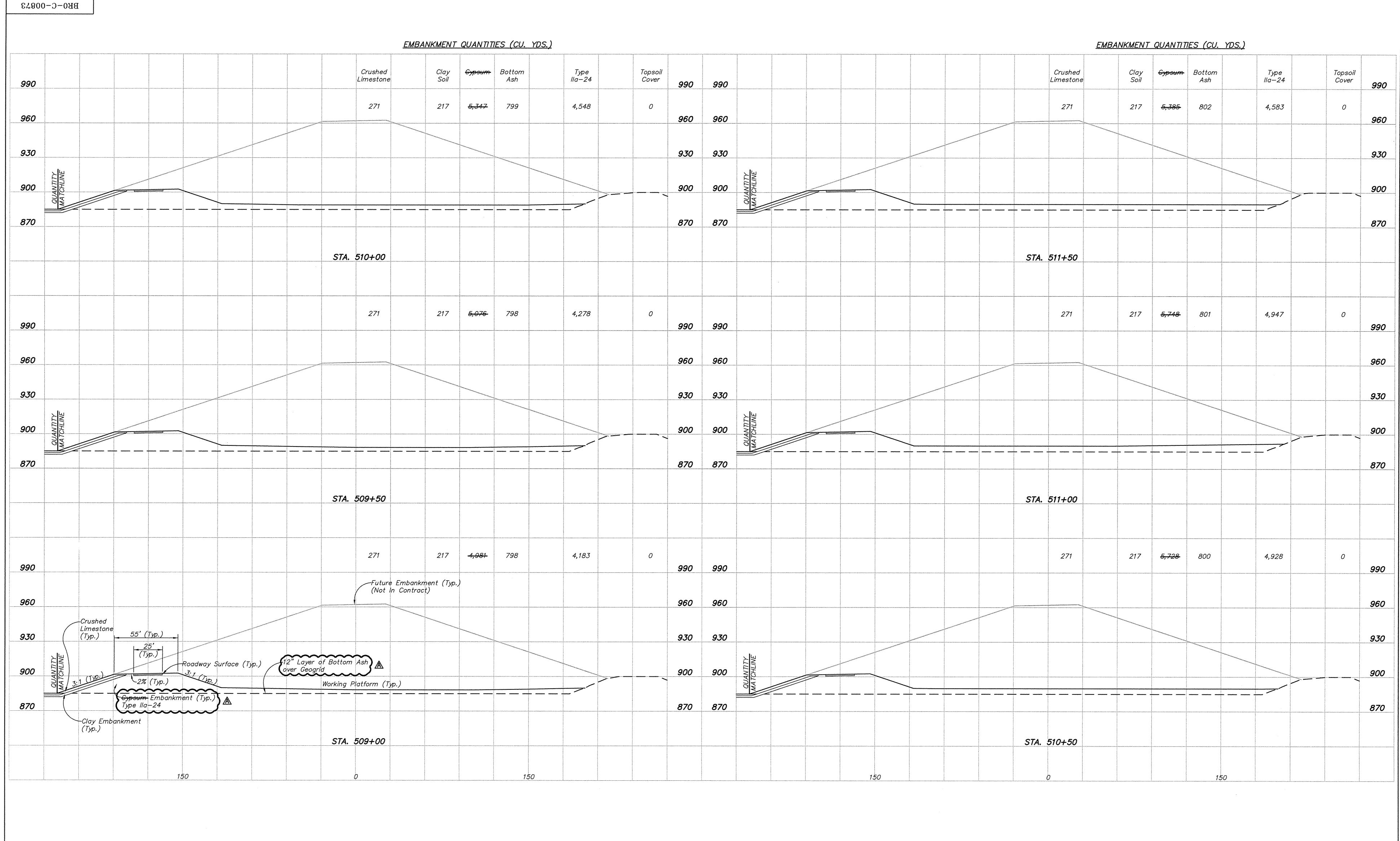
Kentucky an **e∙on** company

ENGINEERS
LECONCTON . LOUISVILLE . CINCINNATI ATLANTA

ST. LOUIS . JEFFERSONALLE . ATLANTA

Drawn: CDH
Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 BR0-C-00872

Drawing No:



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

ARTER DIKE\CONST\XSHT4.DWG

Wossparder

Washada

REVISIONS

Rev. Drawn Date: Drawn By: Revision Mode

A 06-19-08

STA. 509+00 THROUGH STA. 511+50

MAIN ASH POND - STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN

ENGINEERS

LEXINGTON : LOUISVILE : CNCNNATI : COLUNBUS

Scale: AS SHOWN

Drawn: CDH

Date: NOVEMBER, 2007

Checked: VJS/DAB

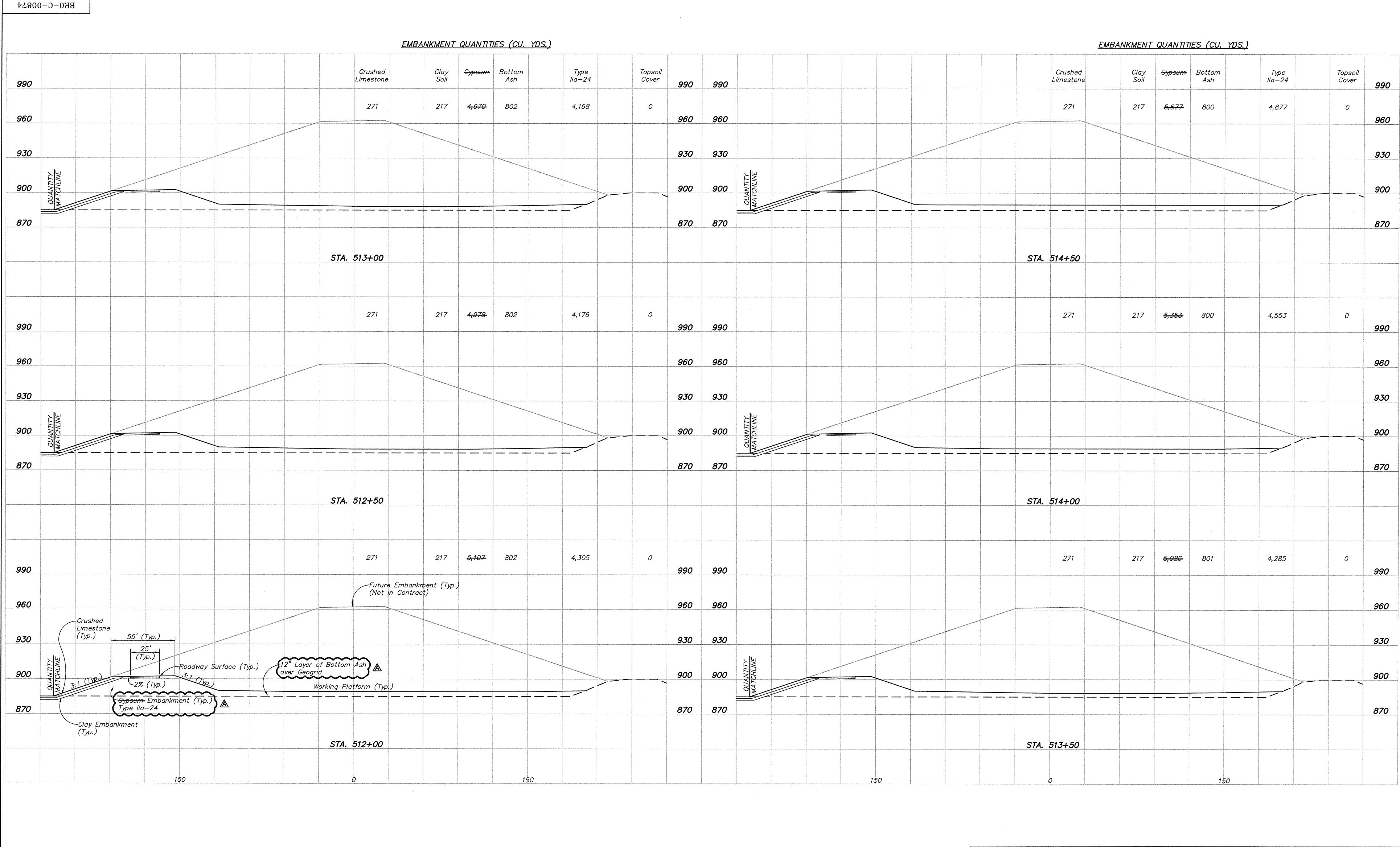
Approved:

JOB NO. JOB NO. JOB NO. JOB NO.

119961

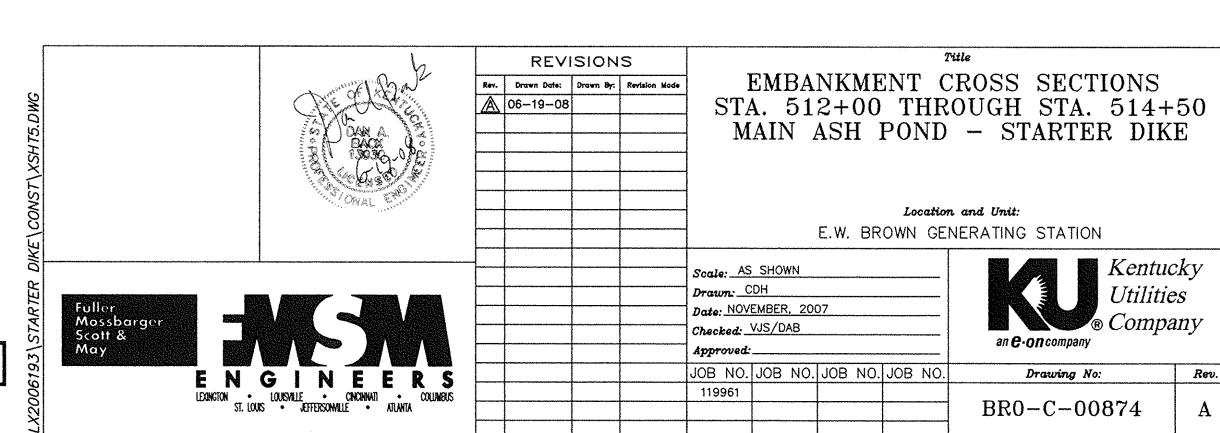
BR0-C-0873 A

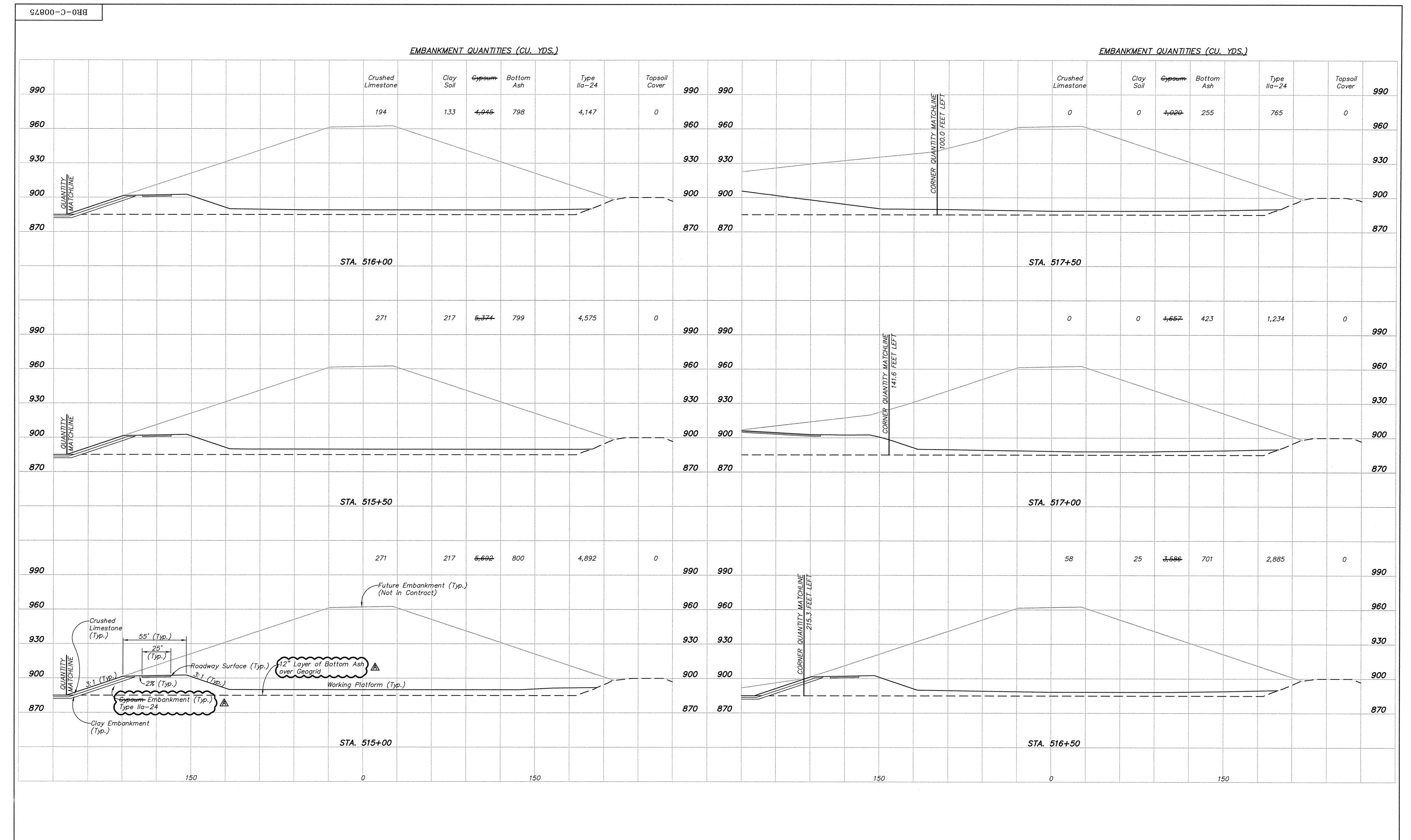
an **e-on** company



2. Future Embankment Outline shown is approximate. Final configuration will be constructed in Phase 5.

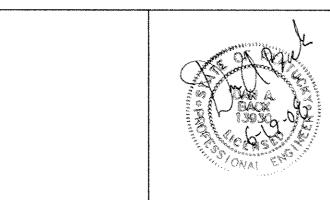
1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.





1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



REVISIONS EMBANKMENT CROSS SECTIONS STA. 515+00 THROUGH STA. 517+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Made A 06-19-08

> Location and Unit: E.W. BROWN GENERATING STATION Kentucky
> Utilities

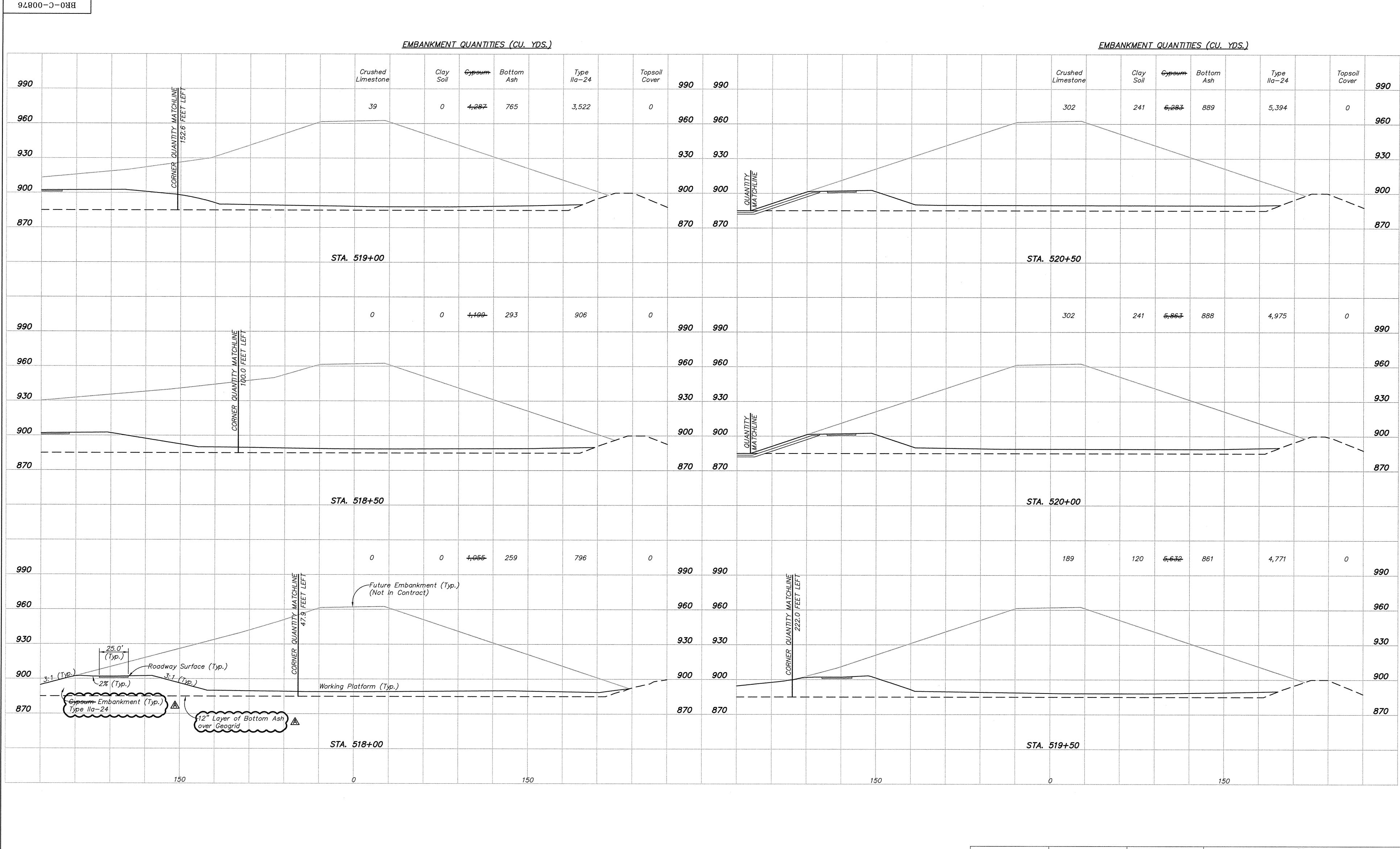
ENGINEERS

LECTRICTION : LOUSVALLE : CHOCHWAT : COLUMBUS

ST. LOUIS : JEFFERSONVILE : ATLANTA

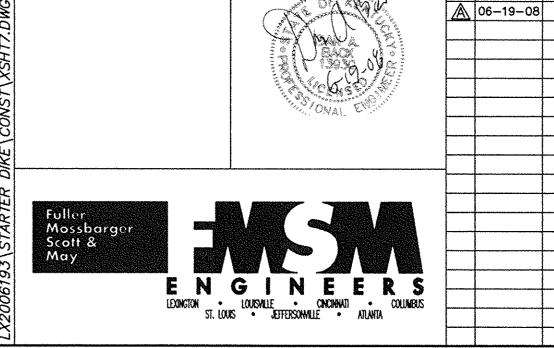
Scale: AS SHOWN Drawn: CDH
Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961

an **e·on** company Drawing No: BR0-C-00875



1. Refer to Typical Sections and drawing BRO—C—00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



REVISIONS

Rev. Drawn Date: Drawn By: Revision Mode

A 06-19-08

STA. 518+00 THROUGH STA. 520+50

MAIN ASH POND - STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN

CDH

Scale: AS SHOWN

Drawn: CDH

Date: NOVEMBER, 2007

Checked: VJS/DAB

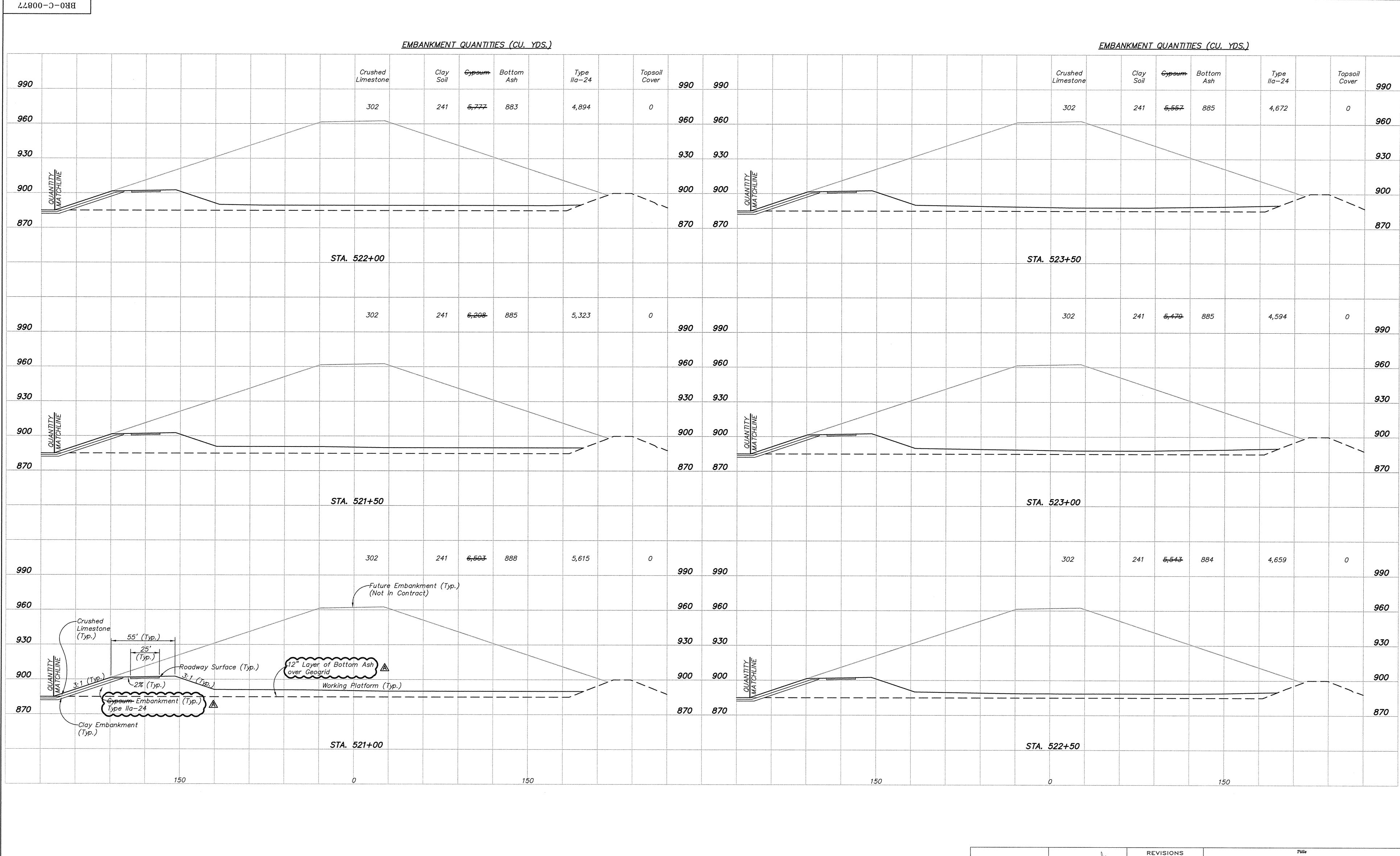
Approved:

JOB NO. JOB NO. JOB NO. JOB NO.

119961

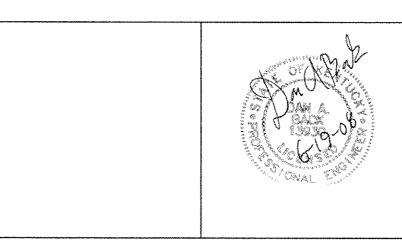
BRO-C-00876

Rev.



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



ENGINEERS
LECONSTON : LOUSVALLE : CINCHINATI : COLUMBUS
ST. LOUIS : JETFERSONALLE : ATLANTA

Rev. Drawn Date: Drawn By: Revision Made

A 06-19-08

S

EMBANKMENT CROSS SECTIONS
STA. 521+00 THROUGH STA. 523+50
MAIN ASH POND - STARTER DIKE

Location and Unit:
E.W. BROWN GENERATING STATION

Scale: AS SHOWN

Drawn: CDH

Date: NOVEMBER, 2007

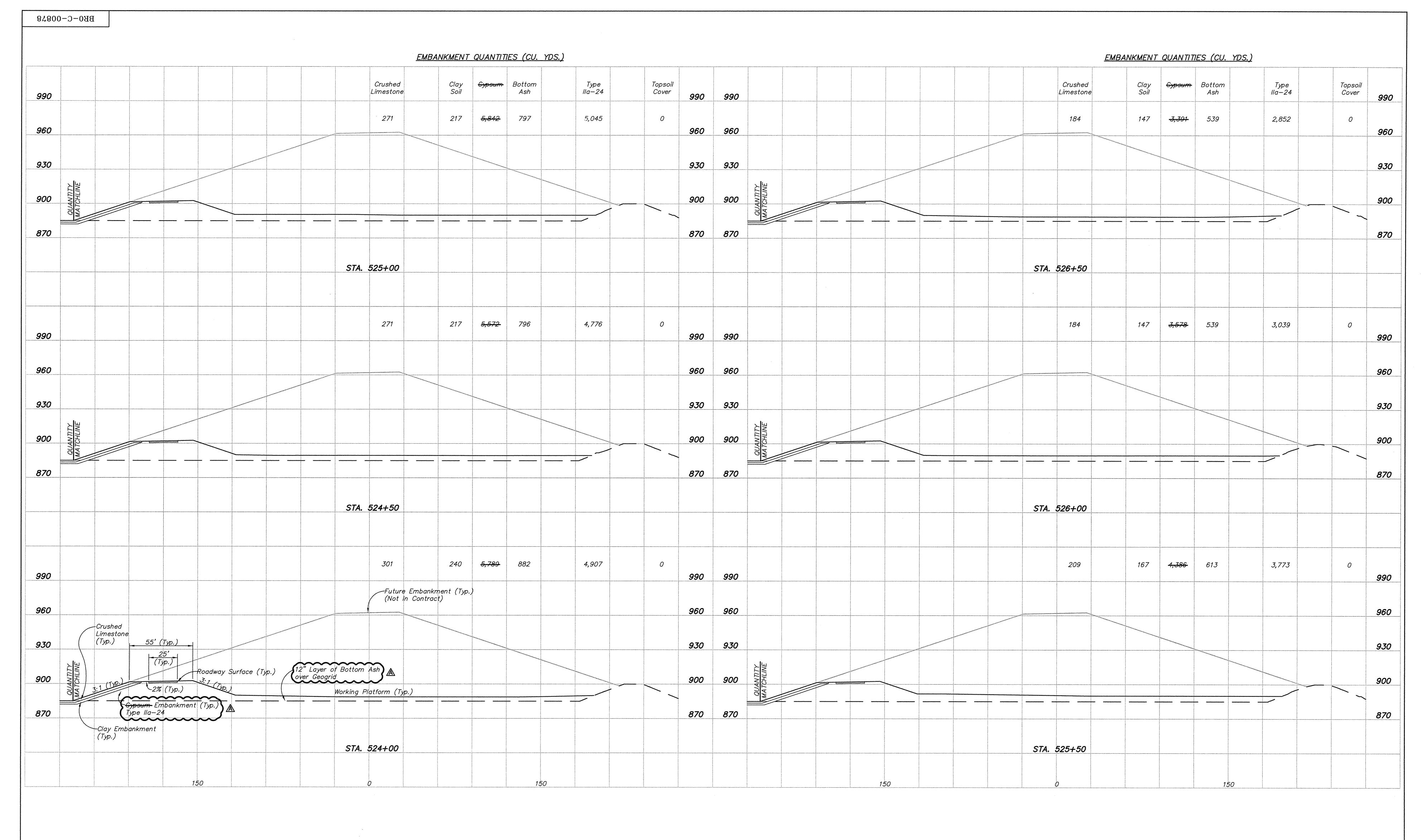
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Approved:

JOB NO. JOB NO. JOB NO. JOB NO. 119961

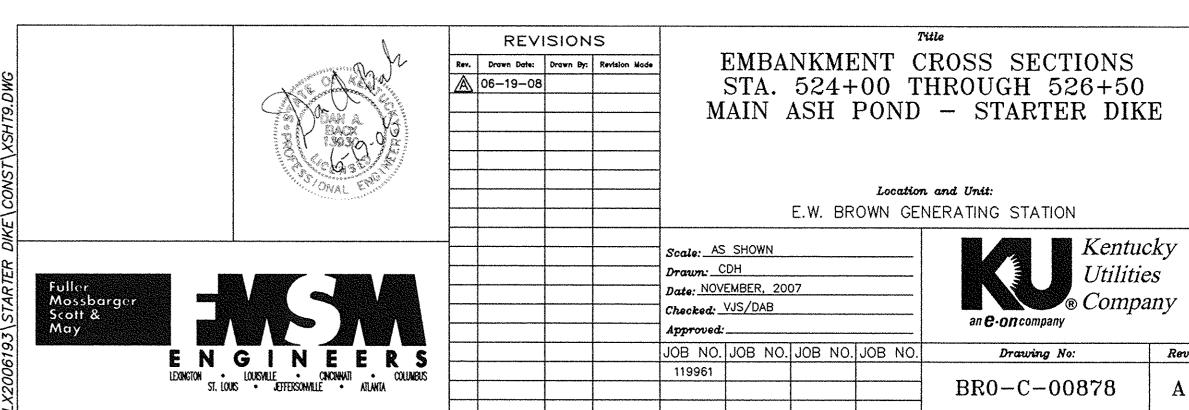
RELEASED FOR CONSTRUCTION - 06/19/08

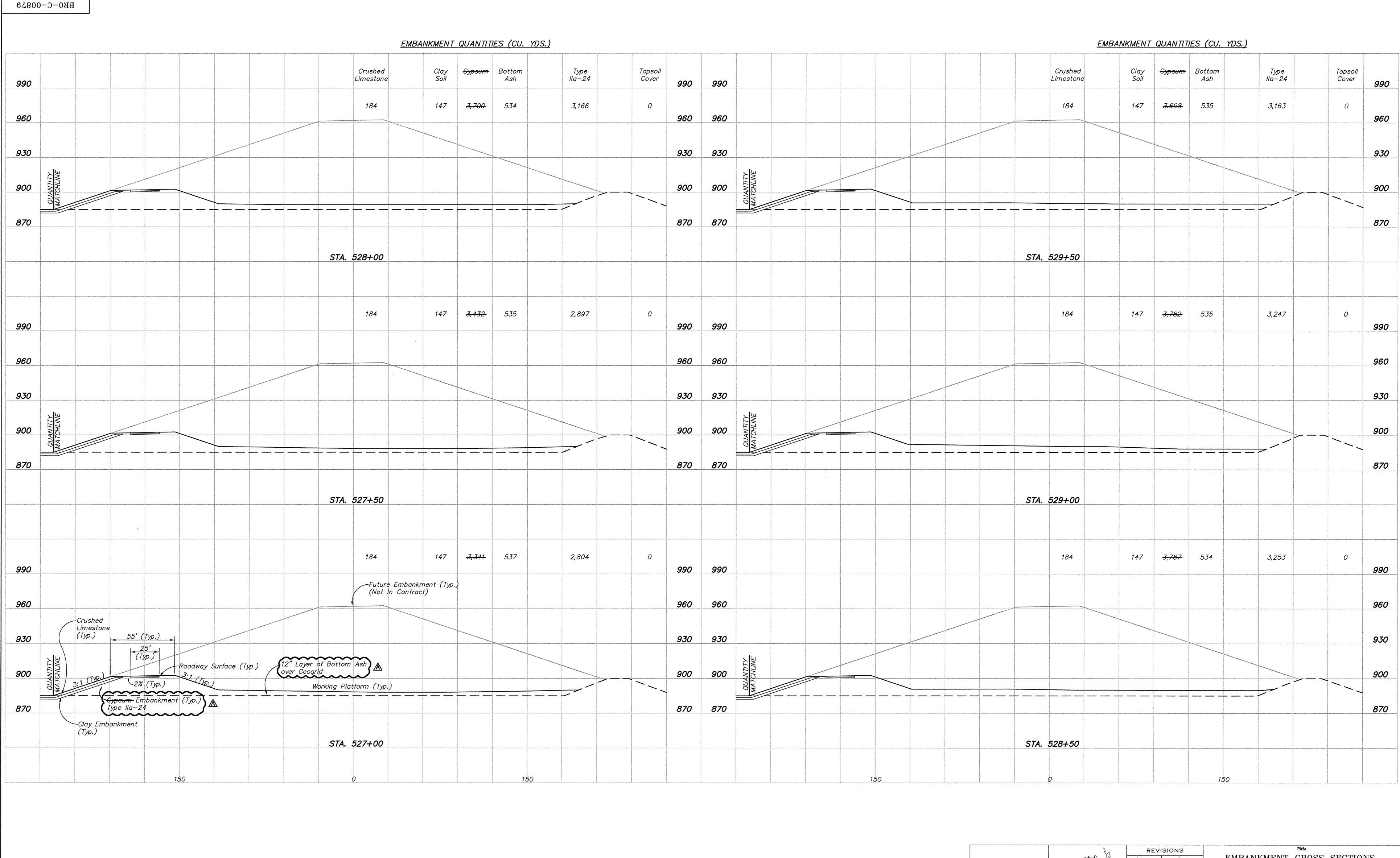
an **e-on** company



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

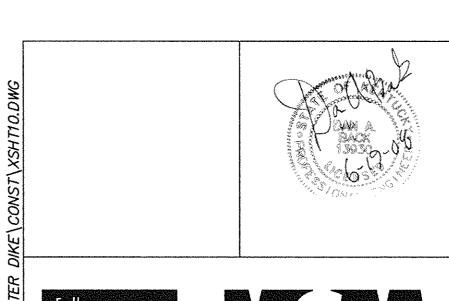
2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.





1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



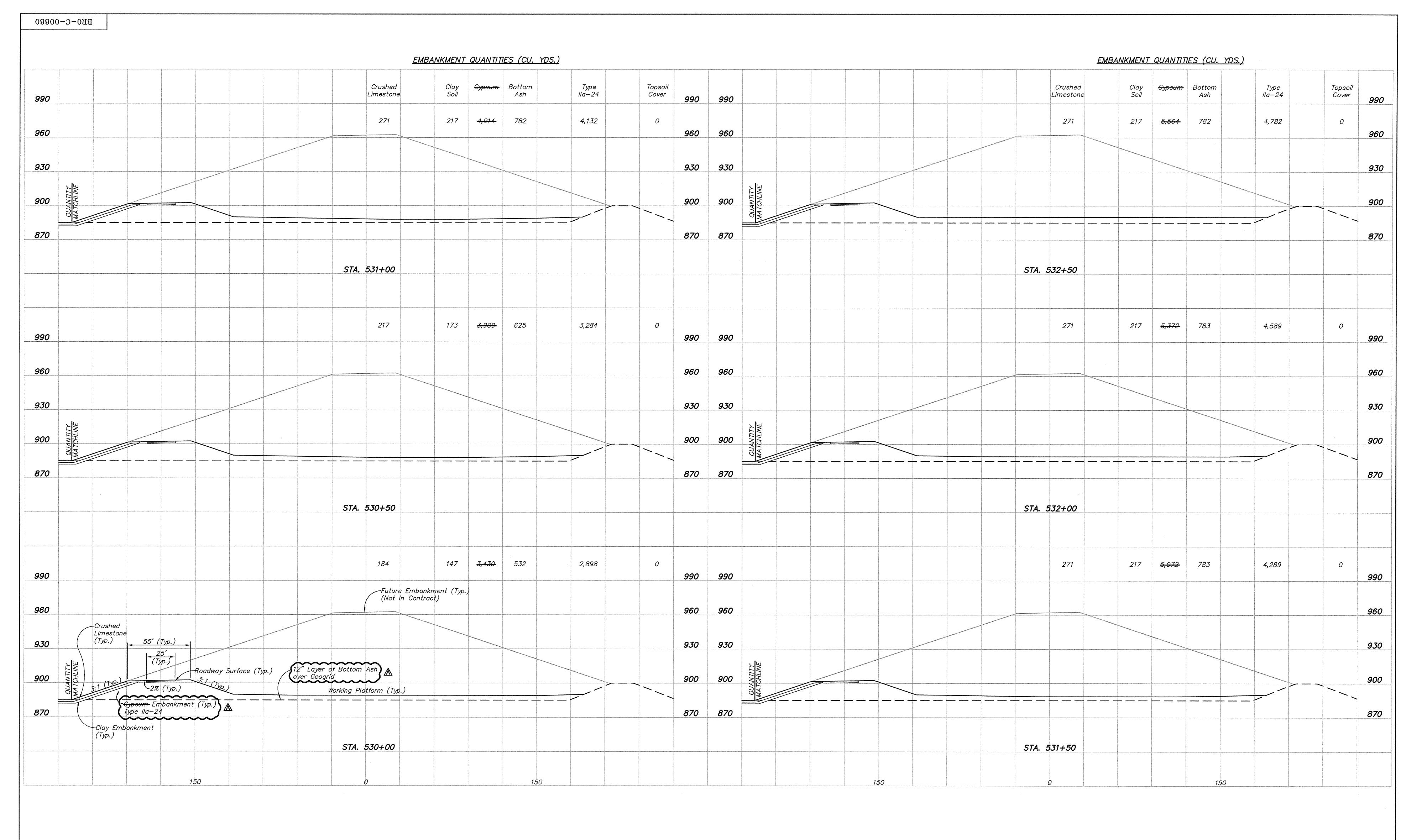
EMBANKMENT CROSS SECTIONS STA. 527+00 THROUGH STA. 529+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Made

A 06-19-08

> Location and Unit: E.W. BROWN GENERATING STATION Scale: AS SHOWN Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB

an **e-on** company Approved:\_\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 Drawing No: BR0-C-00879

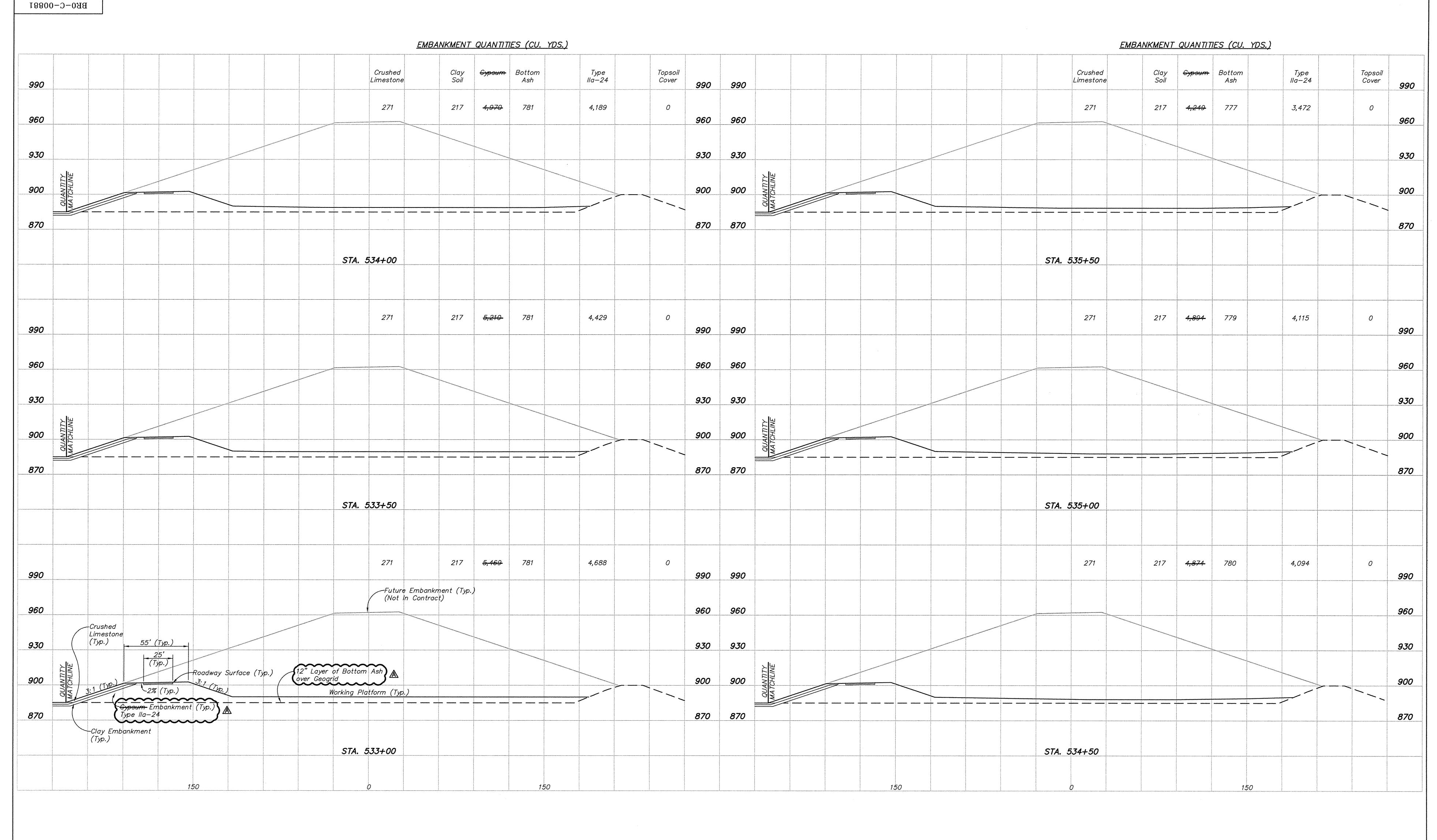
RELEASED FOR CONSTRUCTION - 06/19/08



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate. Final configuration will be constructed in Phase 5.

REVISIONS EMBANKMENT CROSS SECTIONS STA. 530+00 THROUGH 532+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Mode A 06-19-08 Location and Unit: E.W. BROWN GENERATING STATION Scale: AS SHOWN Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB ап **е-оп**сотрапу Approved:\_\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. Drawing No: 119961 BR0-C-00880



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



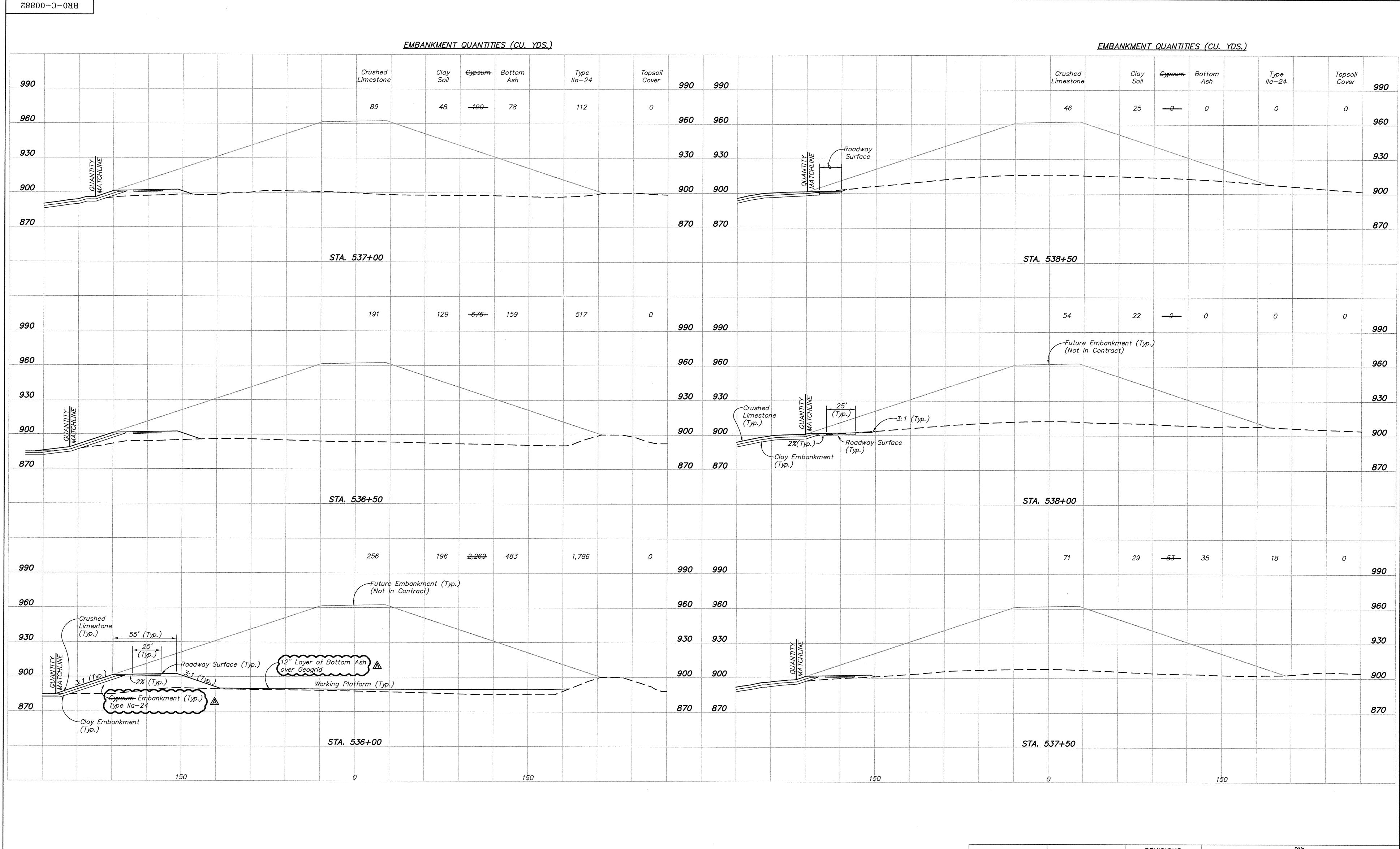
ENGINEERS
LECONCTON : LOUISVILLE : CINCINNATI : COLUMBUS
ST. LOUIS : JETFERSONALLE : ATLANTA

REVISIONS EMBANKMENT CROSS SECTIONS STA. 533+00 THROUGH 535+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Made A 06-19-08

> Location and Unit: E.W. BROWN GENERATING STATION

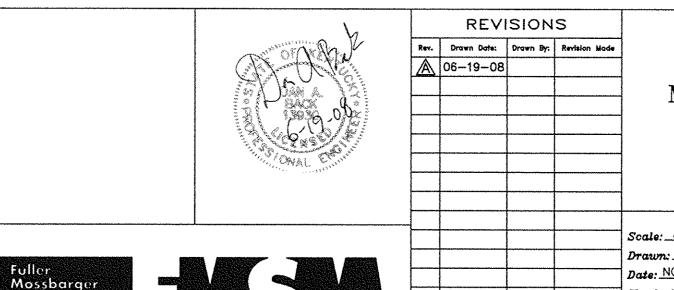
Scale: AS SHOWN Drawn: CDH
Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961

Kentucky an **e∙on** company Drawing No: BR0-C-00881



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



EMBANKMENT CROSS SECTIONS
STA. 536+00 THROUGH 538+50
MAIN ASH POND - STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

E.W. BROWN GEN

Scale: AS SHOWN

Drawn: CDH

Date: NOVEMBER, 2007

Checked: VJS/DAB

Approved:

ENGINEERS

JOB NO. JOB NO. JOB NO. JOB NO.

LECHNICH : LOUISMILE : CNCNNAII : COLUMBIS

ST. LOUIS : JEFFERSONAILE : ATLANTA

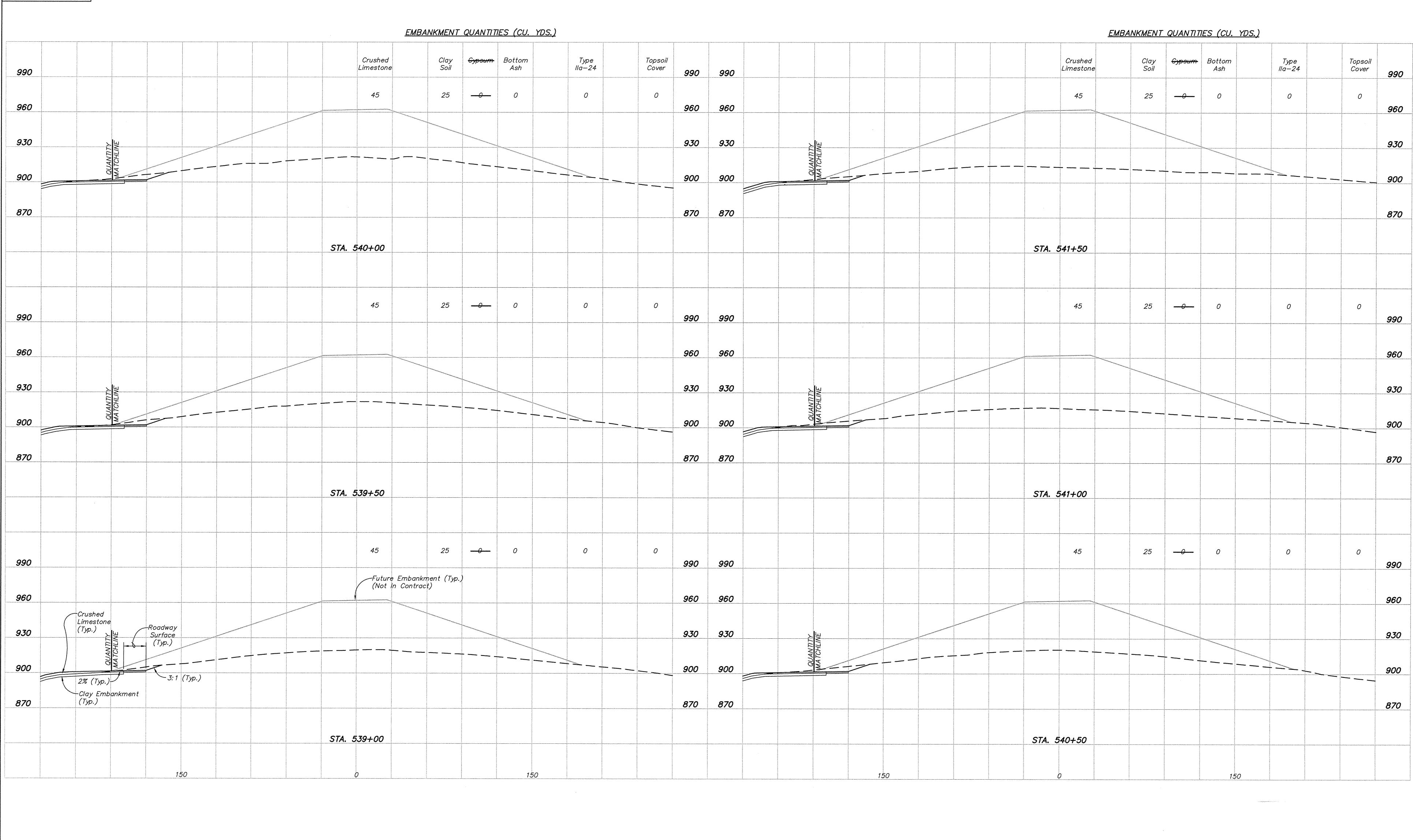
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Kentucky Utilities

an **e∙on** company

Drawing No:

BR0-C-00882



BE0-C-00883

1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

REVISIONS

Rev. Drawn Date: Drawn By: Revision Mode

OG-19-08

EMBANKMENT CROSS SECTIONS
STA. 539+00 THROUGH STA. 541+50
MAIN ASH POND - STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN
Drawn: CDH
Date: NOVEMBER, 2007
Checked: VIS/DAB
Approved:
St. JOB NO. JOB NO. JOB NO. JOB NO. Drawing No: Rev.

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SI. JOB NO. JOB NO. JOB NO. JOB NO. Drawing No: Rev.

119961

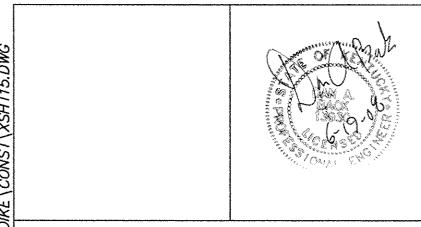
BRO-C-00883

A

BK0-C-00884 EMBANKMENT QUANTITIES (CU. YDS.) EMBANKMENT QUANTITIES (CU. YDS.) Type Ila-24 Topsoil Cover Crushed Type IIa-24 Topsoil Cover Clay Soil Crushed Clay Soil Limestone Limestone 990 990 25 39 960 960 960 930 930 930 930 900 900 900 *870* 870 STA. 543+00 STA. 544+50 25 <del>-0-</del> 990 990 990 960 960 960 930 930 930 930 900 900 870 870 870 STA. 544+00 STA. 542+50 25 66 990 990 Future Embankment (Typ.) (Not In Contract) 960 960 -Roadway 930 930 930 900 900 *870* (Typ.) *870* STA. 542+00 STA. 543+50 REVISIONS EMBANKMENT CROSS SECTIONS STA. 542+00 THROUGH STA. 544+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Mode A 06-19-08

# NOTES:

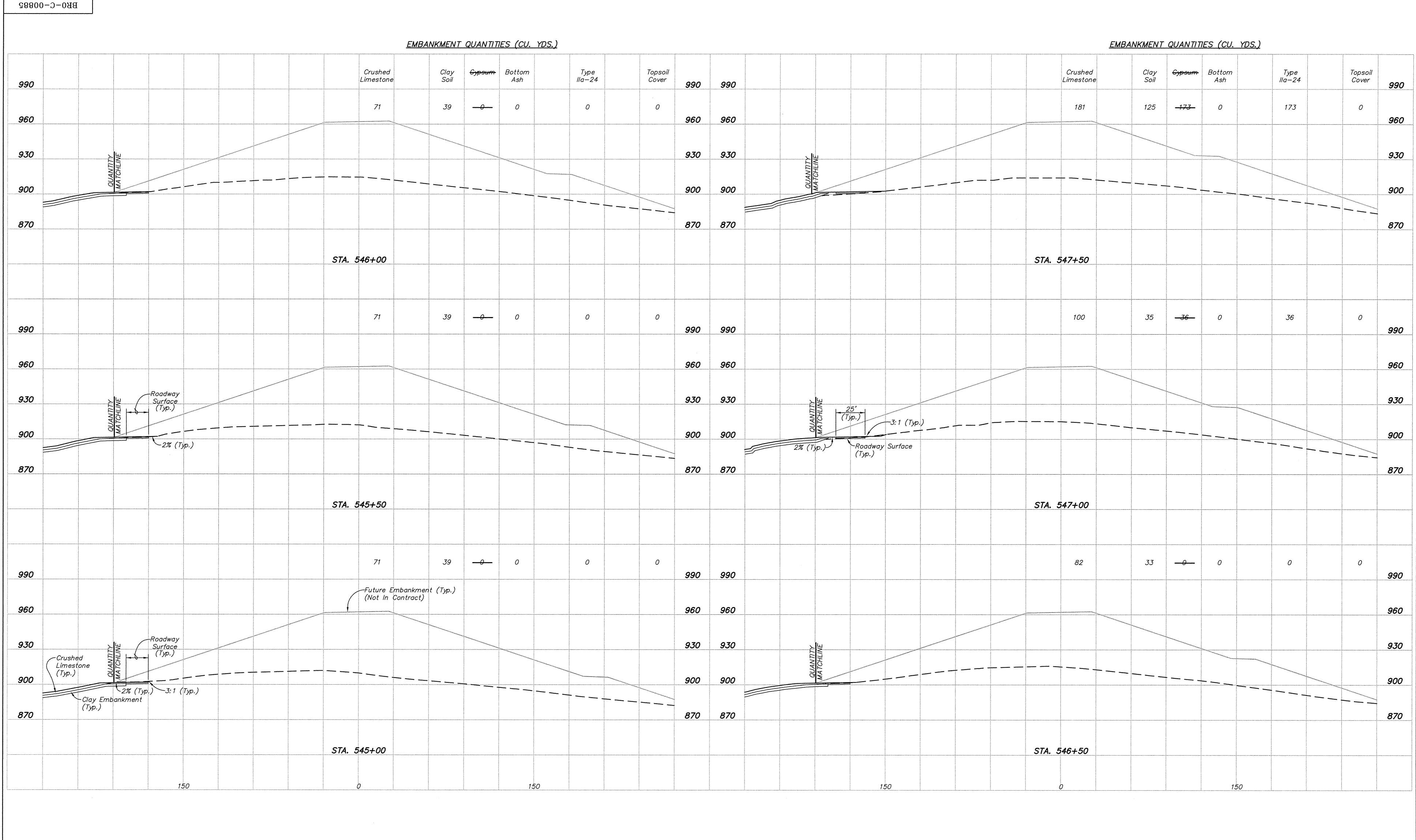
- 1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.
- 2. Future Embankment Outline shown is approximate.
  Final configuration will be constructed in Phase 5.



Location and Unit: E.W. BROWN GENERATING STATION

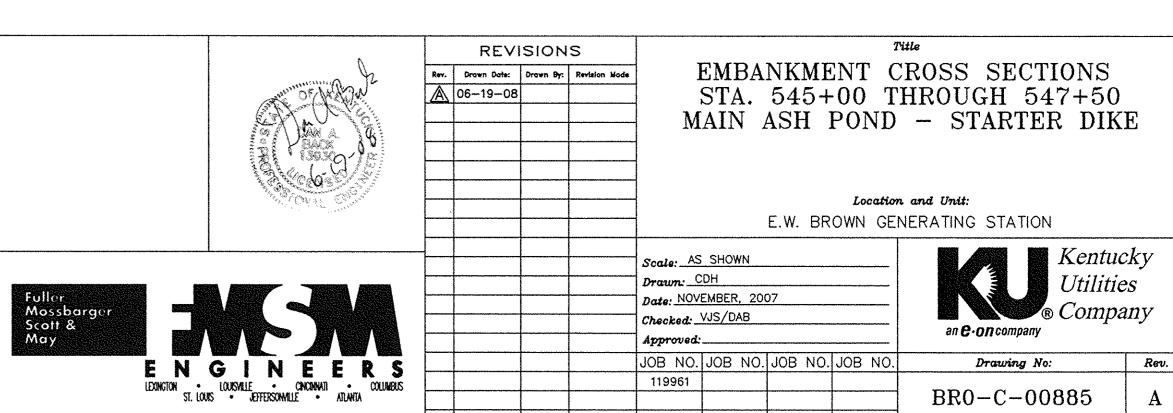
Scale: AS SHOWN Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB an **e•on** company Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 BR0-C-00884

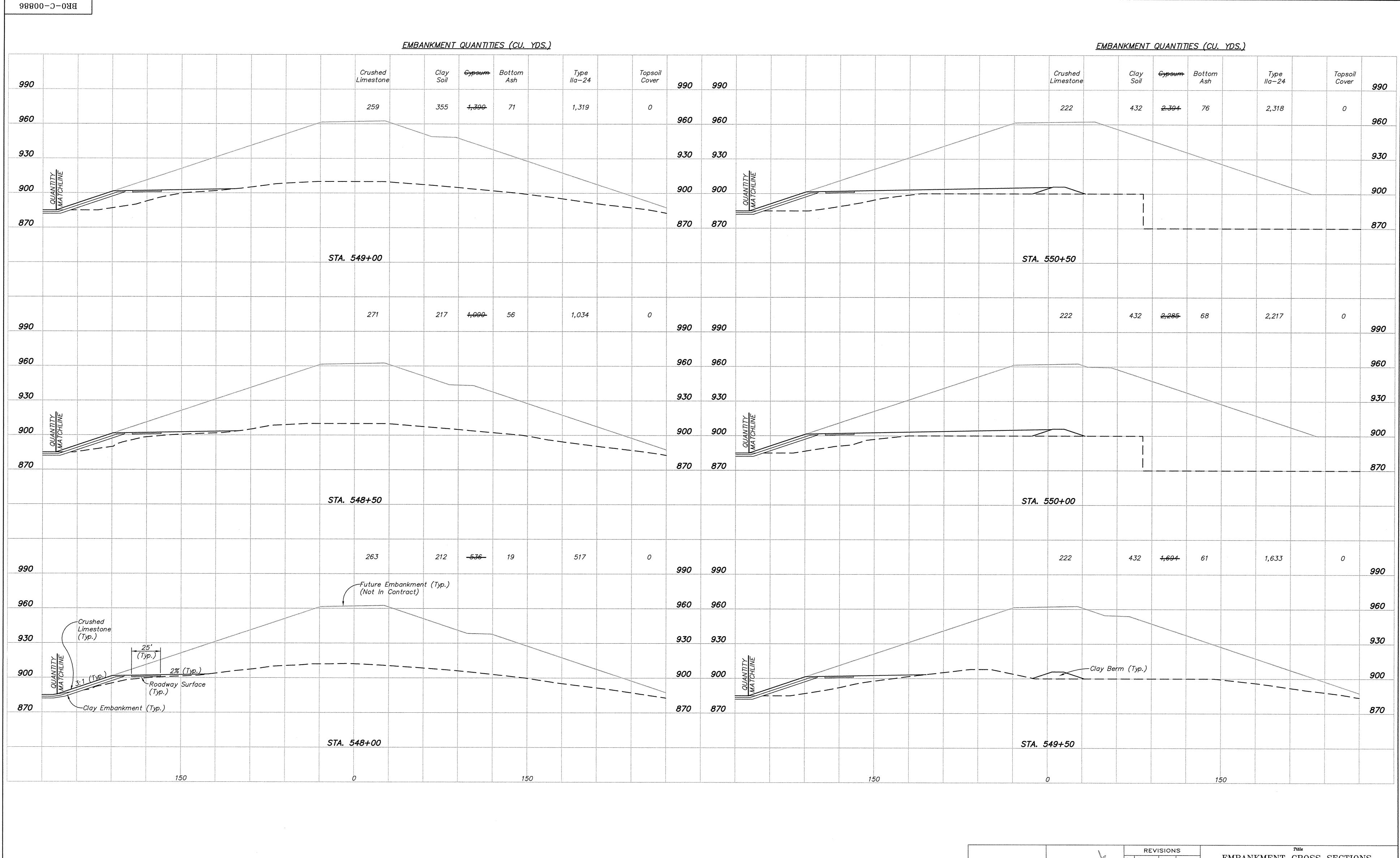
Drawing No:



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

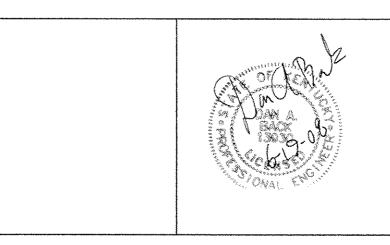
2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.





1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



ENGINEERS
LEGINGTON ST. LOUIS . JETFERSONALLE . ATLANTA

EMBANKMENT CROSS SECTIONS STA. 548+00 THROUGH 550+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Orawn By: Revision Made A 06-19-08

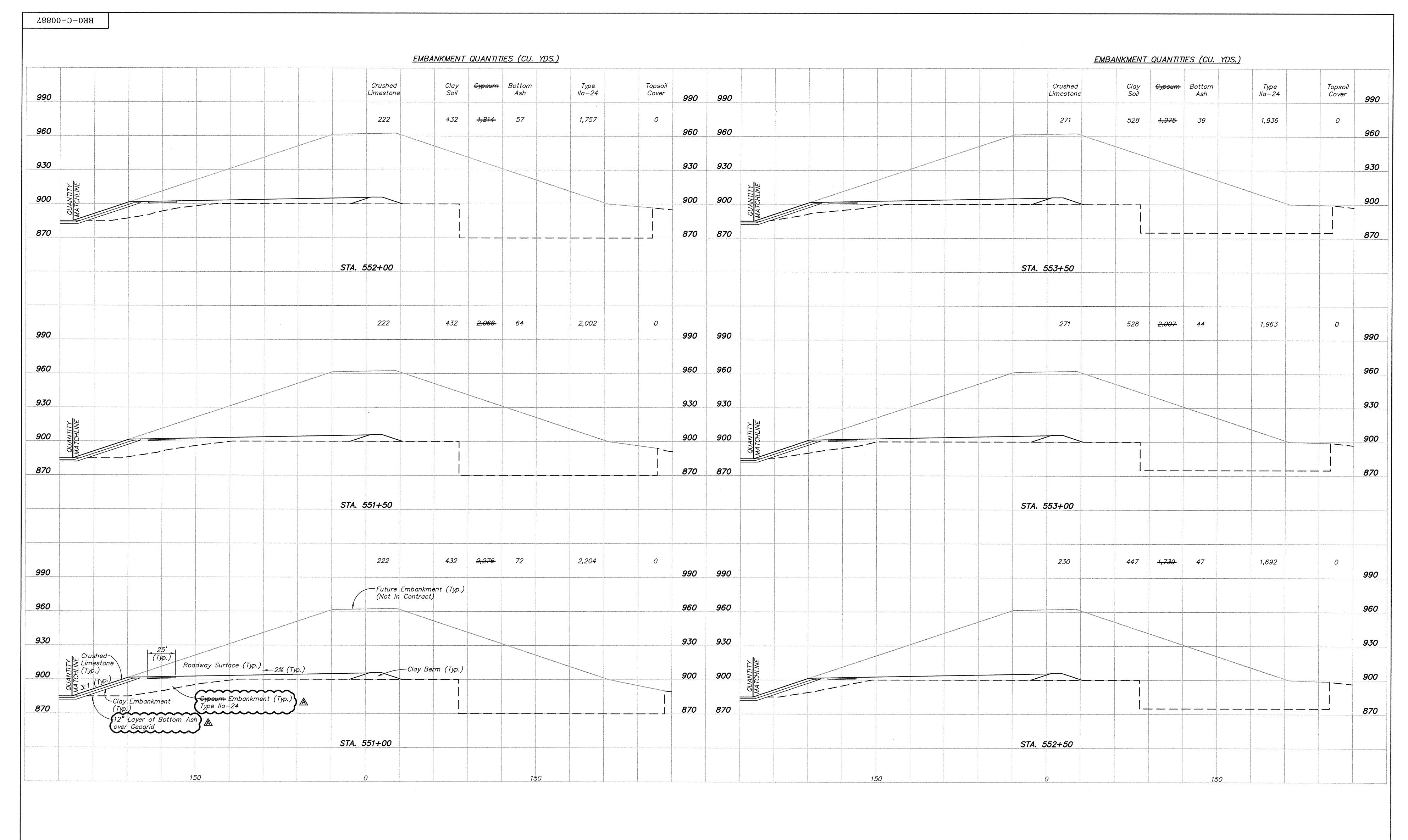
Location and Unit:

E.W. BROWN GENERATING STATION

Scale: AS SHOWN Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. BR0-C-00886

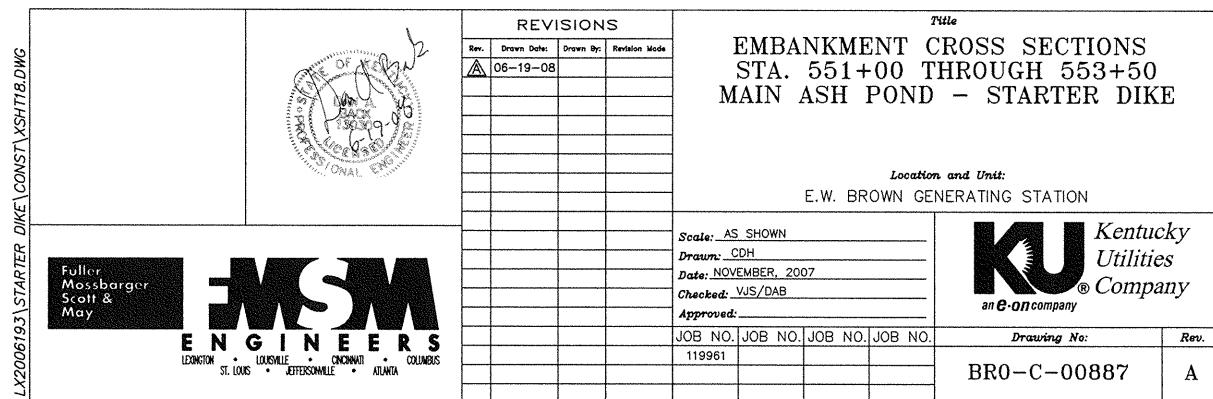
RELEASED FOR CONSTRUCTION - 06/19/08

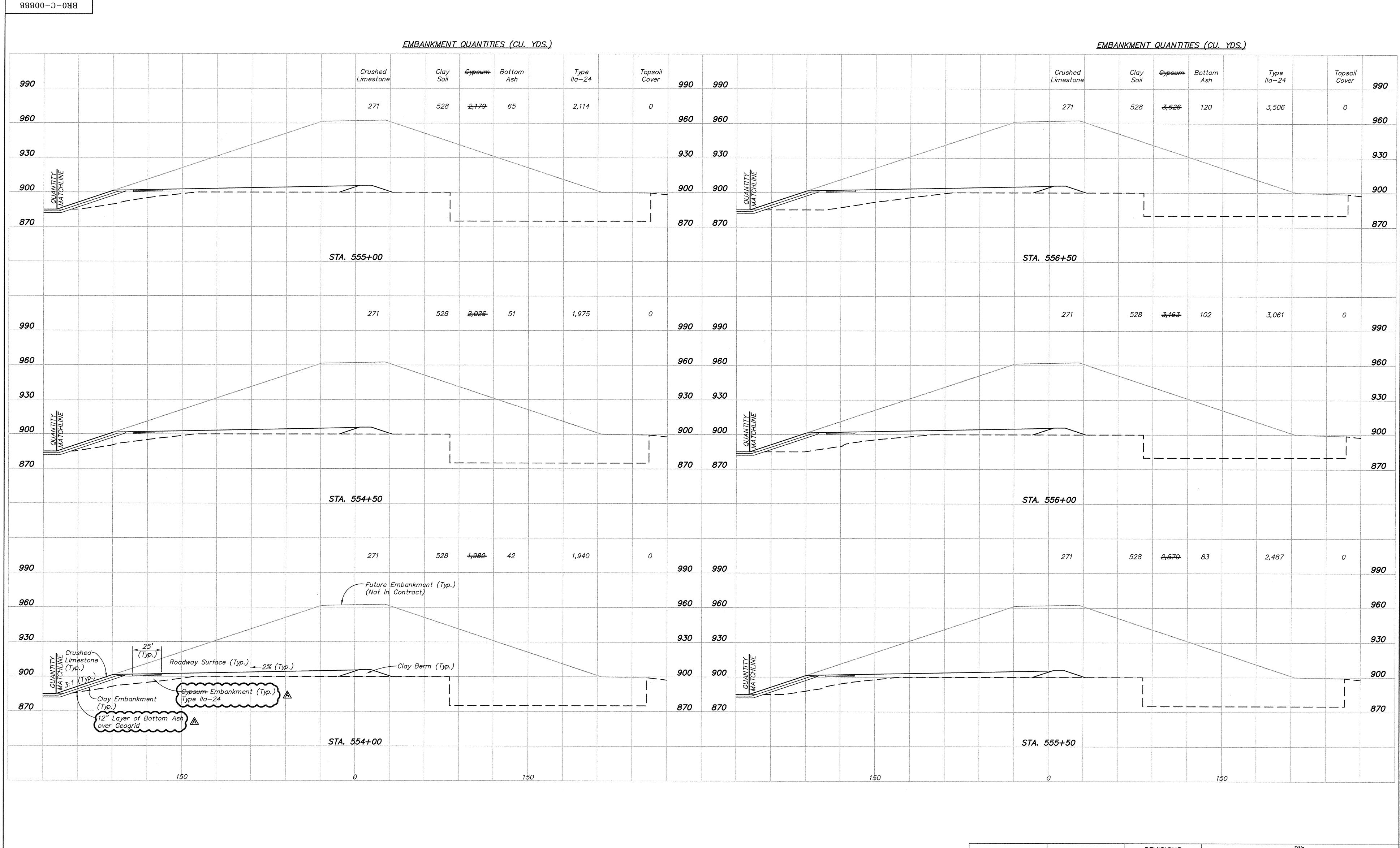
Drawing No:



1. Refer to Typical Sections and drawing BRO—C—00836 for additional information.

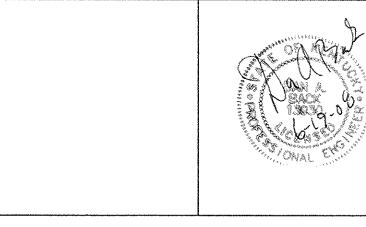
2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.





1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



REVISIONS Rev. Drawn Date: Drawn By: Revision Made A 06-19-08

EMBANKMENT CROSS SECTIONS STA. 554+00 THROUGH STA. 556+50 MAIN ASH POND - STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_\_ ENGINEERS LEXINGTON ST. LOUIS : JEFFERSONMALE : ATLANTA

Scale: AS SHOWN JOB NO. JOB NO. JOB NO. JOB NO.

an **e-on** company Drawing No: BR0-C-00888

RELEASED FOR CONSTRUCTION - 06/19/08

BB0-C-00888 EMBANKMENT QUANTITIES (CU. YDS.) EMBANKMENT QUANTITIES (CU. YDS.) Type IIa-24 Topsoil Cover Crushed Clay Soil Type IIa-24 Topsoil Cover Clay Soil Crushed Limestone Limestone 990 271 4,422 4,792 271 960 960 960 930 930 930 930 900 870 870 870 STA. 558+00 STA. 559+50 271 4,049 271 4,826 990 990 960 960 930 930 900 900 870 870 870 STA. 557+50 STA. 559+00 <del>3,895</del> 271 3,751 *528* 271 528 204 4,698 990 990 990 —Future Embankment (Typ.) (Not In Contract) 960 960 960 960 930 930 930 930 ... Crushed~ -Roadway Surface (Typ.) \_\_\_\_2% (Typ.) —Clay Berm (Typ.) 900 900 900 -Clay Embankment 870 870 STA. 557+00 STA. 558+50 REVISIONS Rev. Drawn Date: Drawn By: Revision Mode

# NOTES

1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

Fuller
Mossbarger
Scott &
May

ENGINEER S

LEXINGTON . LOUISVILE . CHORNATI . COLUMBUS

ST. LOUIS LEXINGTON . COLUMBUS

ST. LOUIS . ATLANTA

EMBANKMENT CROSS SECTIONS

STA. 557+00 THROUGH 559+50

MAIN ASH POND — STARTER DIKE

Location and Unit:

E.W. BROWN GENERATING STATION

10: AS SHOWN

Scale: AS SHOWN

Drawn: CDH

Date: NOVEMBER, 2007

Checked: VJS/DAB

Approved:

JOB NO. JOB NO. JOB NO. JOB NO.

119961

BRO-C-00889

Kentucky

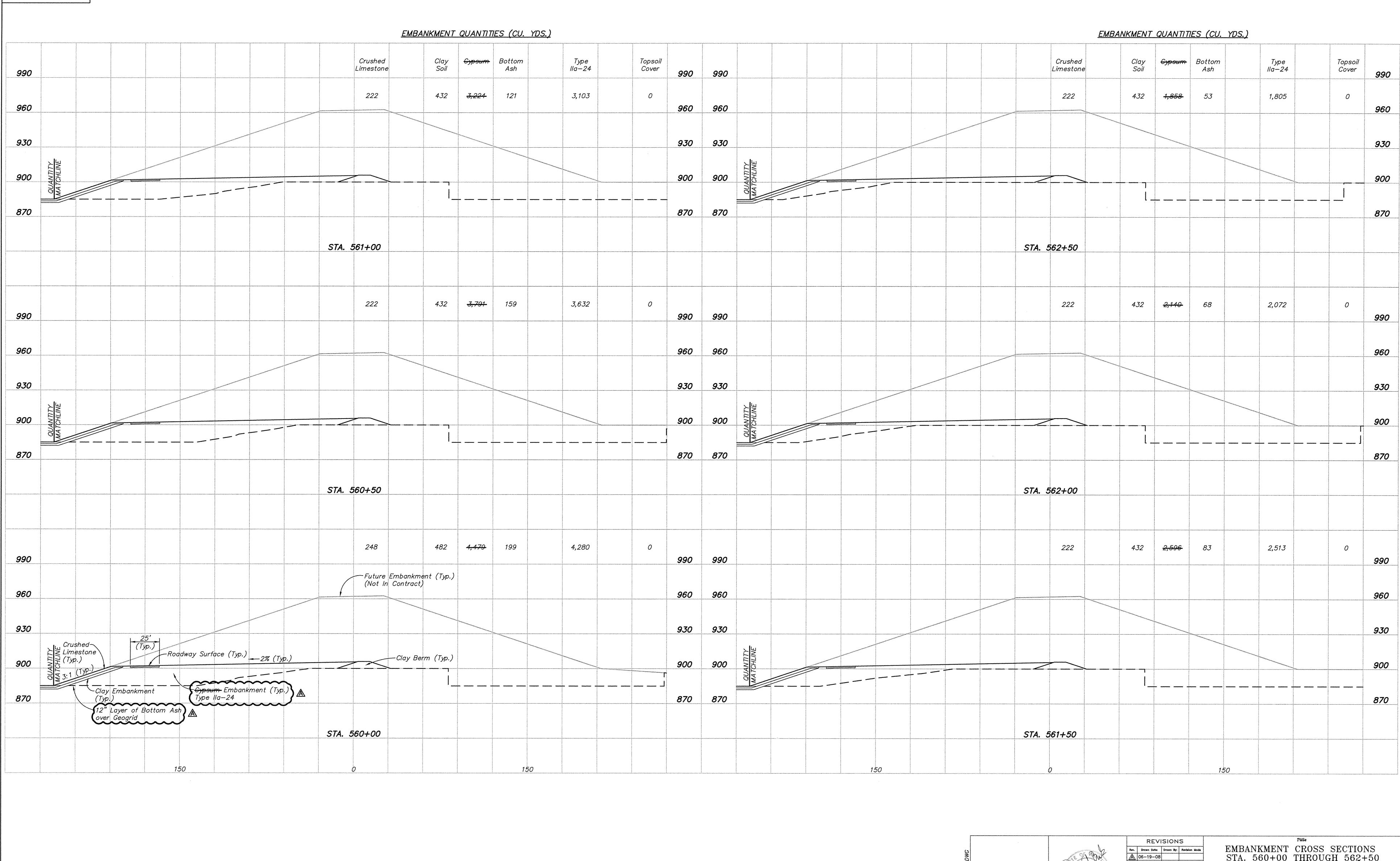
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BB0-C-00880

1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate. Final configuration will be constructed in Phase 5.

EMBANKMENT CROSS SECTIONS STA. 560+00 THROUGH 562+50 MAIN ASH POND - STARTER DIKE

Location and Unit: E.W. BROWN GENERATING STATION

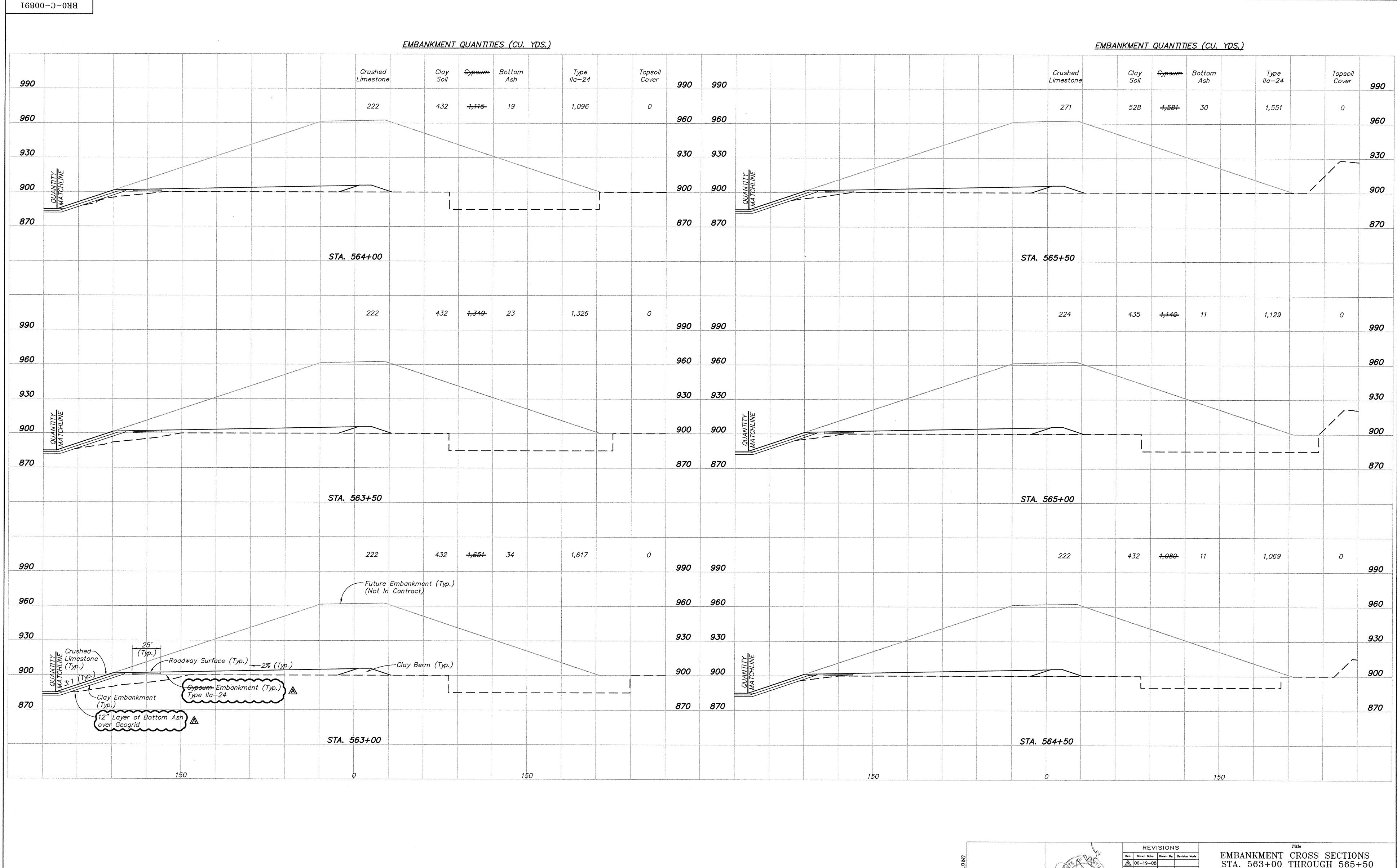
Scale: AS SHOWN Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_\_

an **e-on** company Drawing No:

RELEASED FOR CONSTRUCTION - 06/19/08 ENGINE ERS
LEGINGTON ST. LOUISVALLE COLUMBUS

LEGINGTON ST. LOUISVALLE COLUMBUS

LEGINGTON ST. LOUISVALLE COLUMBUS



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

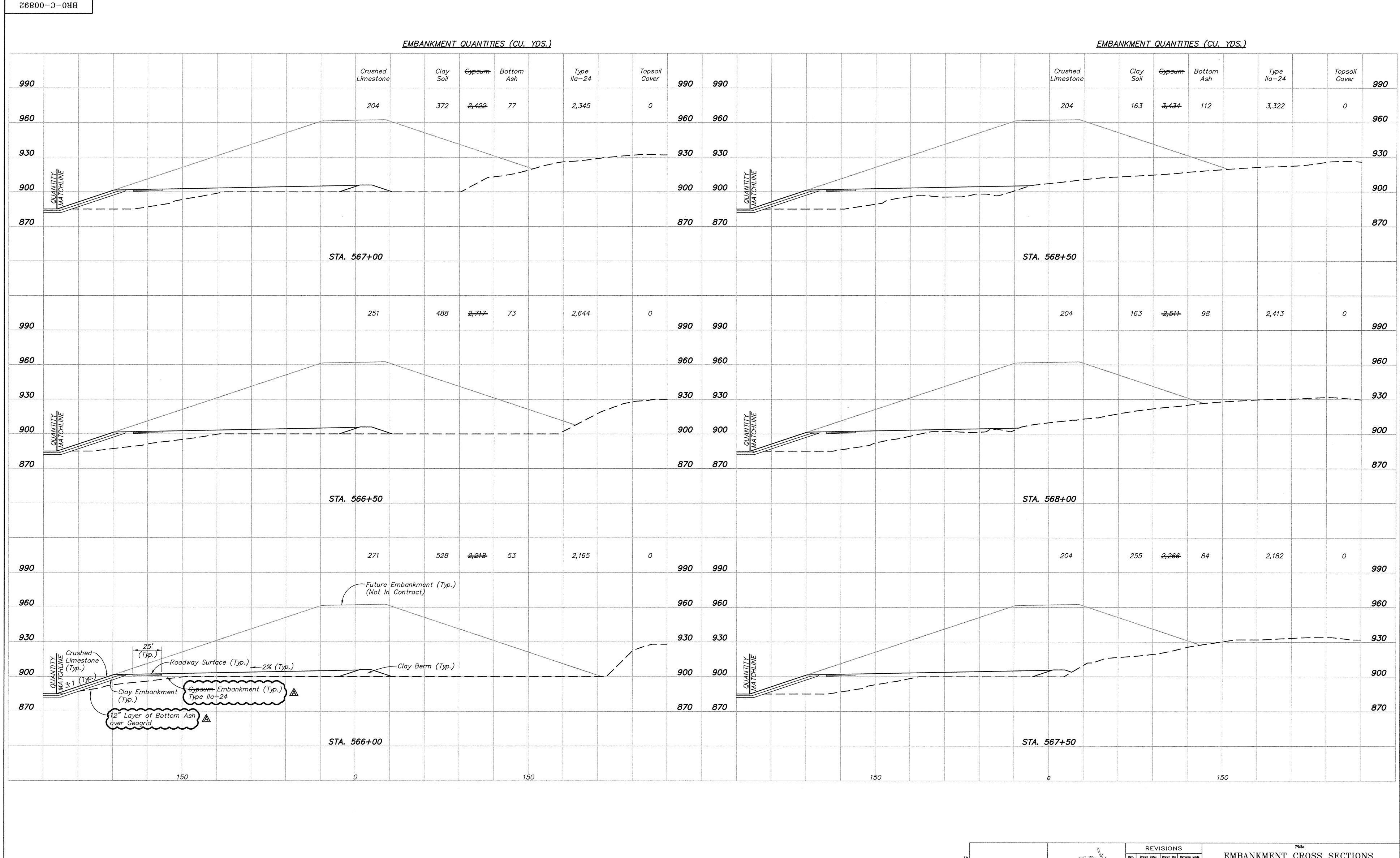
TO VALUE OF THE PARTY OF THE PA ENGINE ERS
LEGINGTON : LOUSSMILE : CNCHNATI : COLUMBUS
ST. LOUIS : JEFFERSONVILE : ATLAVIA EMBANKMENT CROSS SECTIONS STA. 563+00 THROUGH 565+50 MAIN ASH POND - STARTER DIKE

Location and Unit: E.W. BROWN GENERATING STATION

Scale: AS SHOWN Drawn: CDH
Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961

Kentucky an **e·on** company Drawing No: BR0-C-00891

RELEASED FOR CONSTRUCTION - 06/19/08



NOTES: Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

Rev. Drawn Date: Drawn By: Revision Made ▲ 06-19-08

EMBANKMENT CROSS SECTIONS STA. 566+00 THROUGH 568+50 MAIN ASH POND - STARTER DIKE

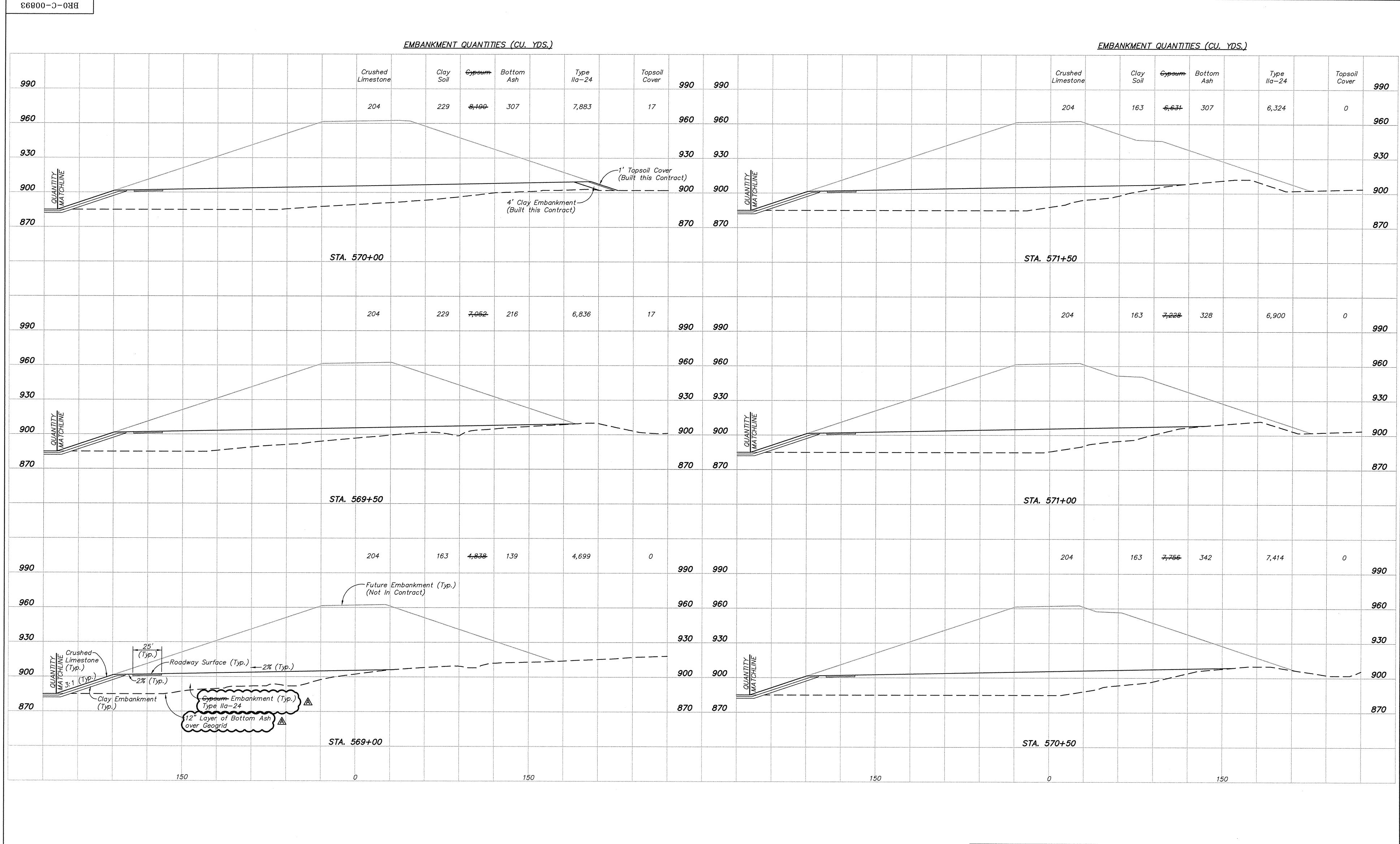
Location and Unit: E.W. BROWN GENERATING STATION

Drawing No:

an **e-on** company

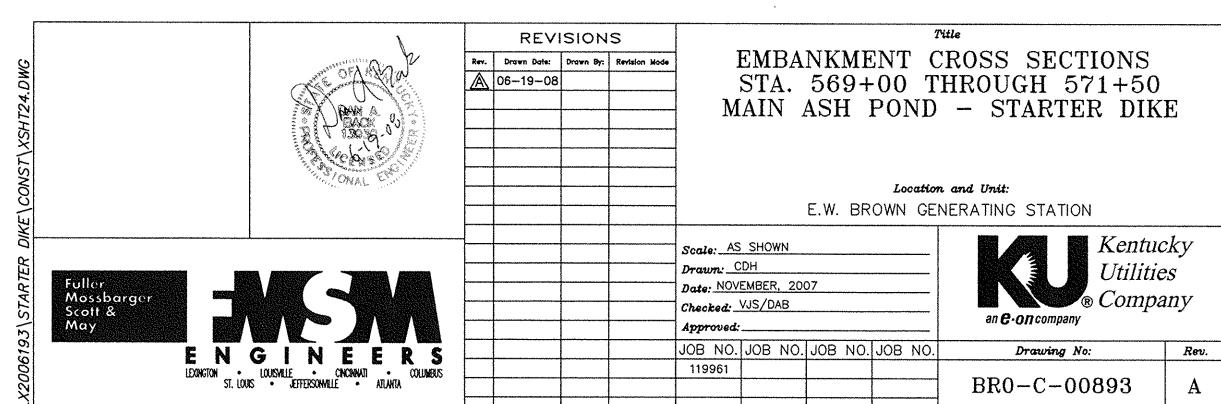
Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 BR0-C-00892

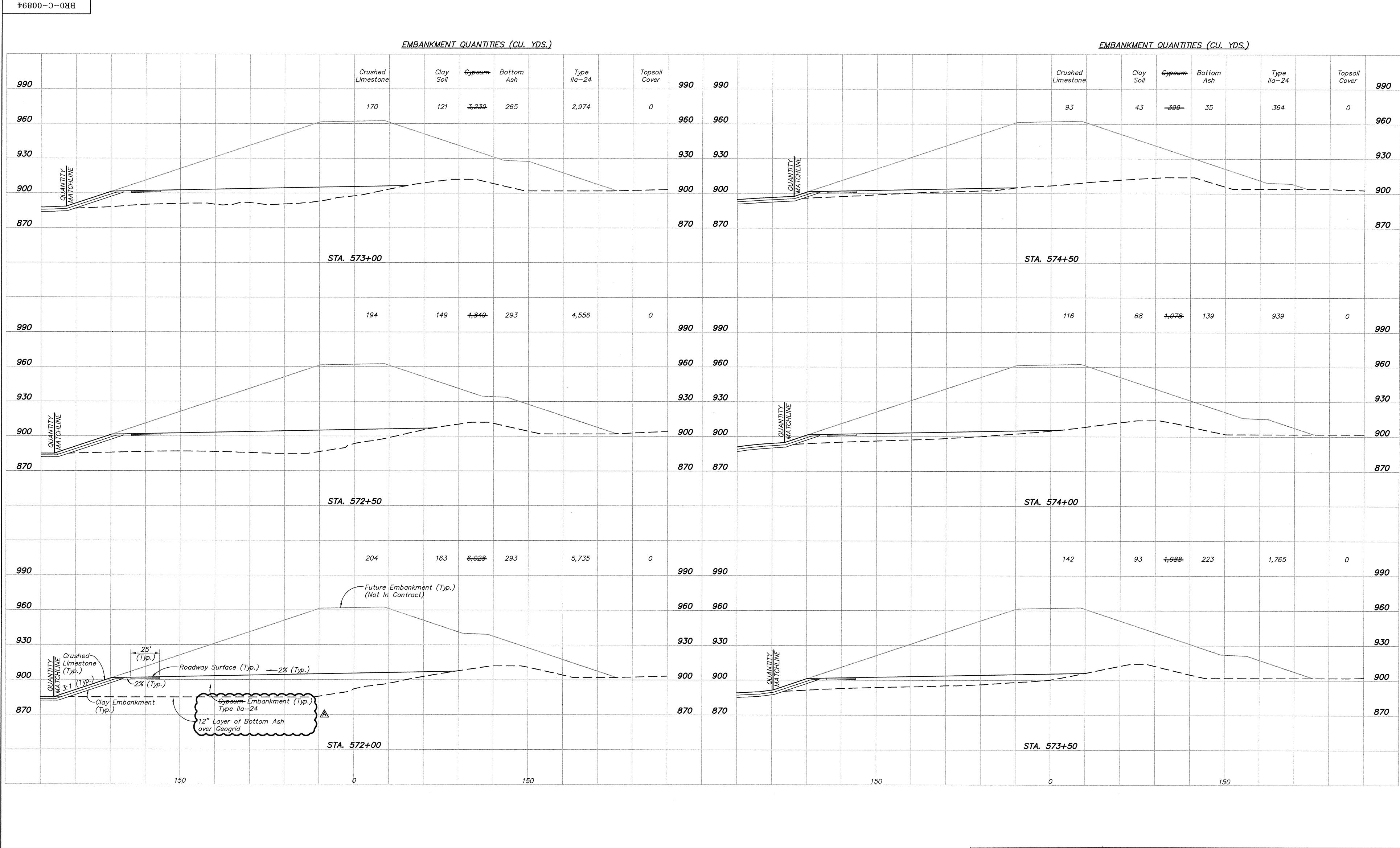
RELEASED FOR CONSTRUCTION - 06/19/08



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

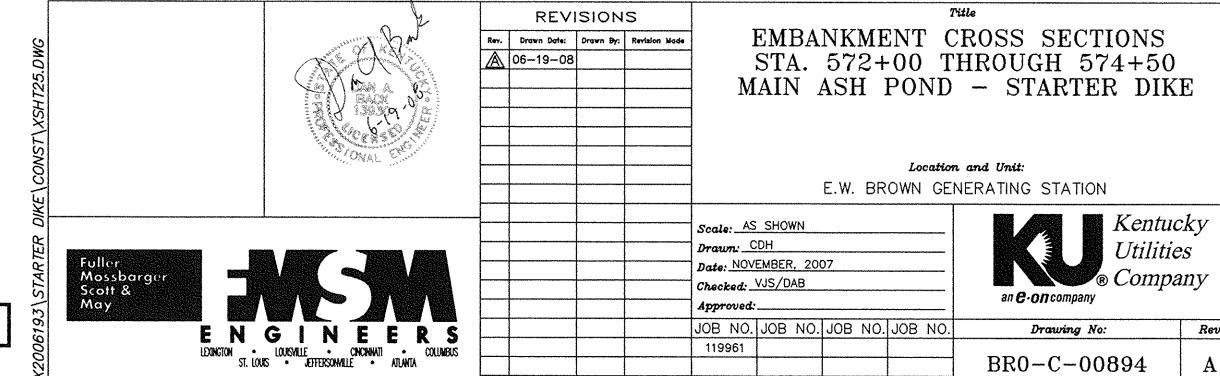
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Final configuration will be constructed in Phase 5.



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2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

ENGINEERS COLUMBUS

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EMBANKMENT CROSS SECTIONS STA. 575+00 THROUGH 577+50 MAIN ASH POND - STARTER DIKE

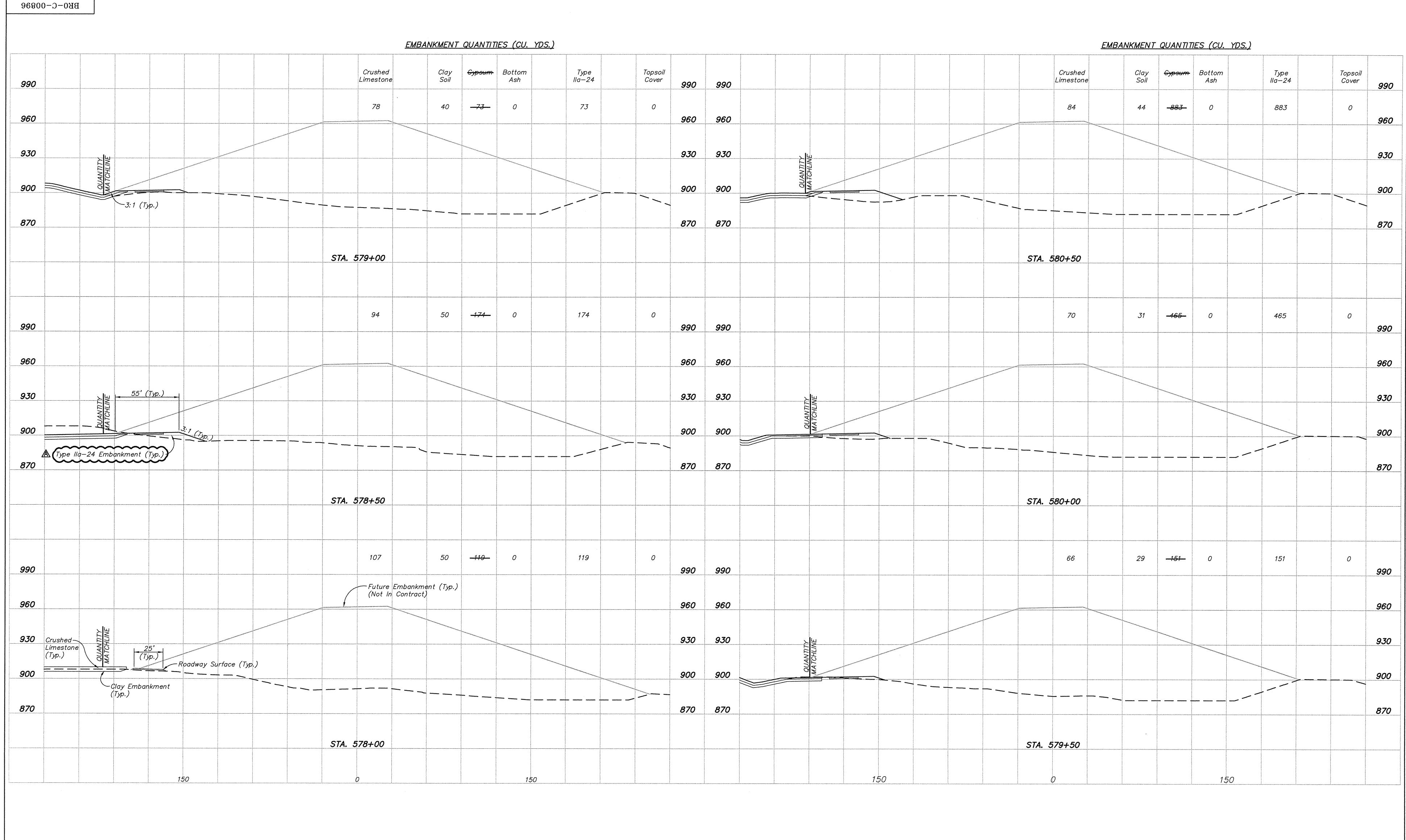
Location and Unit: E.W. BROWN GENERATING STATION

Drawn: CDH
Date: NOVEMBER, 2007 Checked: VJS/DAB an **e∙on** company Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961

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DMS Version 2.0

Drawing No:



1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.

REVISIONS EMBANKMENT CROSS SECTIONS STA. 578+00 THROUGH STA. 580+50 MAIN ASH POND - STARTER DIKE Rev. Drawn Date: Drawn By: Revision Mode <u>A</u> 06-19-08

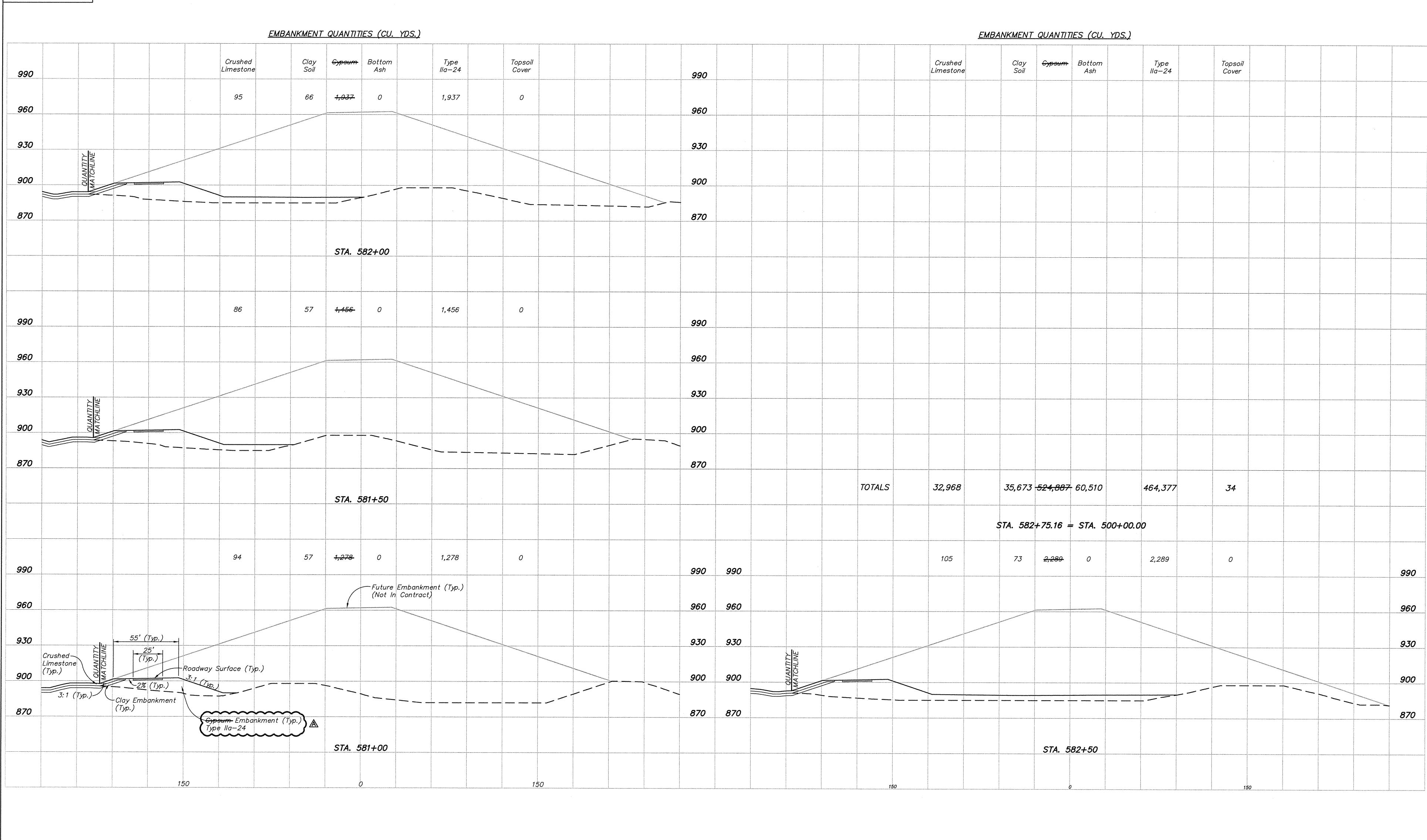
> Location and Unit: E.W. BROWN GENERATING STATION

Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB

an **e-on** company Approved:\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 Drawing No: BR0-C-00896

RELEASED FOR CONSTRUCTION - 06/19/08

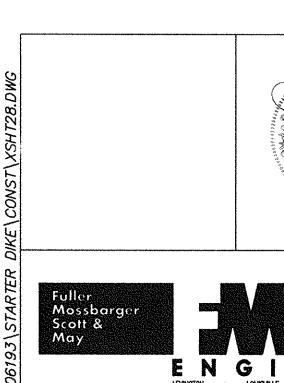
Rev.



BB0-C-0083

1. Refer to Typical Sections and drawing BRO-C-00836 for additional information.

2. Future Embankment Outline shown is approximate.
Final configuration will be constructed in Phase 5.



REVISIONS Rev. Drawn Date: Drawn By: Revision Made A 06-19-08

EMBANKMENT CROSS SECTIONS STA. 581+00 THROUGH 582+50 MAIN ASH POND - STARTER DIKE

Location and Unit: E.W. BROWN GENERATING STATION

ENGINE COLUMBUS

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LECTRICION ST. LOUIS . LOUIS

Scale: AS SHOWN Drawn: CDH Date: NOVEMBER, 2007 Checked: VJS/DAB an **e-on** company Approved:\_\_\_\_ JOB NO. JOB NO. JOB NO. JOB NO. 119961 BR0-C-00897

RELEASED FOR CONSTRUCTION - 06/19/08

Drawing No:



STEVEN L. BESHEAR GOVERNOR

LEONARD K. PETERS
SECRETARY

#### **ENERGY AND ENVIRONMENT CABINET**

DEPARTMENT FOR ENVIRONMENTAL PROTECTION
DIVISION OF WATER
200 FAIR OAKS, 4TH FLOOR
FRANKFORT, KENTUCKY 40601
www.kentucky.gov

#### CERTIFICATE OF INSPECTION FOR DAM AND APPURTENANT WORKS

**Note:** The Division of Water does not intend this report to be taken as an assurance that no other problems exist at this site or that this dam is safe. The reports sole intent is to provide you a factual account of the conditions observed at the site during the inspection. If you have questions, write this office at the above listed address or call (502) 564-3410.

ID of Dam: 0737 Hazard Class: HIGH

Name of Dam: KENTUCKY UTILITIES Owner: KY Utilities Co

FLY ASH DAM (2) EW Brown Generating Station

Agency Interest: 3148

County:MercerAddress:815 Dix Dam RdInspection Date:July 30, 2008City:Harrodsburg

 Weather:
 70 Deg, Sunny
 State:
 KY

 Phone:
 859-748-4401

**Inspection Type:** Dams

Persons Present at Inspection: Gary Wells, Scott Phelps

Height of Dam:126 feetNormal Pool Elevation (MSL): 868'Latitude Dec Deg:37.786389Current Pool Elevation (MSL): 868'Longitude Dec Deg:-84.716944Emer. Spillway Elevation (MSL): NA

**Type of Dam:** Earthfill & Rockfill embankment 2175 ft long with a top width of 20 ft. 5' berm on downstream side.

Side slopes are both 2:1.

**Upstream Slope of Dam:** Rock cover without vegetation. No signs of major erosion. Minor erosion found near the spillway inlet. No signs of slumps, slides, subsidence, or cracking.

Crest of Dam: Gravel road without vegetation. No signs of cracking or subsidence.

Downstream Slope of Dam: Rock cover without any vegetation. No signs of animal burrows, slides, slumps, or cracking.

Toe Drains: NA



## CERTIFICATE OF INSPECTION FOR

**KY ID: 0737** 

**Principal Spillway:** Existing 24" diameter drop inlet. No outlet works indicated for new structure on plans supplied by

**Principal Spillway Comment:** Catwalk extending out to inlet but no signs of problems.

Stilling Basin: New basin as part of KY 1213 construction so it was in good condition.

**Emergency Spillway:** No spillway indicated on plans for the revised structure. Water is pumped into structure & pumped out to control pool level.

**Emergency Spillway Comments:** No spillway indicated on plans for the revised structure. Water is pumped into structure & pumped out to control pool level.

Drawdown System: NA

**Location of Drawdown Valve:** None **Last Date of Operation:** NA

Does Hazard Classification need to be Reevaluated? The existing Main Ash Pond is now being classified as a Class C - High Hazard Dam in accordance with 401 KAR 4:030. With the construction of the new Auxiliary Pond, the Main Ash Pond shares an adjacent embankment with Aux Pond. The Aux Pond is High hazard because of residences downstream. If failure of the adjacent embankment occurs, then both dams are a hazard to downstream residences. The Main Ash Pond expansion will be constructed without an emergency spillway. A request to waiver the emergency spillway was given by KYDOW on March 8, 2007, provided the PMP design storm event can be safely routed through the new principal spillway riser structure during each phase of Main Ash Pond expansion project. The SITES results show that the PMP storm does not overtop the new crest elevations during each phase. The minimum crest elevations for Phases 1 through 5 are 902', 912', 928', 946', and 962'. The maximum water surface elevations based on the PMP design storm for those corresponding crest elevations are 900.6, 910.6, 926.7, 944.8, and 960.5 respectively.

#### Were Photographs Taken? Yes

#### **General Comments and Recommendations:**

KY 737 has not changed from the previous inspection. Main Ash Pond is undergoing a vertical expansion in five phases to the final elevation of 962.0' and has 6700 ac-ft of storage capacity. Phase I is the starter dike construction and is to begin in the latter half of 2008.

**Inspector:** Gary Wells, PE

**Reviewer:** Gary Wells, PE **Date:** 10/3/08

## DAM ASSESSMENT FORM



Name of Professional Conducting Inspection: Mark J. Schuhmann P.E.					KY Professional License No.: 12,500		
Company Name: ATC Associates, Inc.					Phone:502-722-1401		
Address: 132 Citiz	ens Blvd. Simpsonvil	le, KY 40067		ALLES TALLS			
Inspection Prepara	tion: Reviewed all pe	rtinent technical doc	cument	tation related to thi	is dam and site in:		
	es 🛛 No 🗌 ; and O	wner's Files: Yes ⊵	✓ No [				
Comments: Due to weather cor	nditions at time of As	sessment in Januar				rformed	
Dam/Pond Name: E. W. Brown N	Main Ash Pond	Hazard Rating: High		ographic Quad: more	Date of Inspection: 1/19/09		
State Dam ID: 737	County: Mercer	Latitude 37°47.1813'			Last Inspection: 7-30-2008		
	ne: E. W. Brown Stat				Owner Phone: 859-	748-4456	
	Dam Road, Harrodsb				- Carried House, and the same		
Site Contact: Tom				Phone:	territoria de la constanti		
Drainage Area (AC): 126	Surface Area(AC) 126	Height (Ft): 126		Crest Length (Ft): ~4700	Crest Width (Ft): 41 to 64	Crest Elevation(Ft): 900	
Slope: Upstream: 2:1 Downstream: 2:1	Principal Spillway Type: Drop Inlet						
CCP placed in Pond: Fly ash, bottom ash, pyrites, process water drainage	Emergency Spillw Type: Open Chan		e:	Spillway Control Elevation: Not Available	Freeboard(Ft): 7 (estimated)		
	IONS OBSERVED						
Ash Exposed: Yes	s: None:	Location: No water by staked burlap	in pon	d, ash covered	Max. Height above p	ool	
Water Level (Bel	ow Dam Crest, Ft): N		mpound	led			
Ground Moisture	Condition: Dry	Wet Snow	cover	Other:	The Constitution	A factor	
Monitoring: Yes		Gage Rod   Piez	zometer	rs Seepage W	eirs 🔲 Survey Mon	numents 🔲 Othe	
Comments: Piezometers on no	orth end dike not oper	ating, No data provi	ided.				
A UPSTRI	OPE Sinkho	les Appears To	oo Stee	ep 🔲 Depressio	e	Cracks	
ACCEPTABLE DEFICIENT POOR	Comments:						
B CF	☐ Not W	ide Enough Lo	and a set with the side	as Misaligni	rosion	Sinkholes te Surface Drainag	
ACCEPTABLE DEFICIENT	Comments:	Small washout area	a under nch was	sprinkler line note	d on east embankmer regularities noted in v	nt. Small depressi vidth of crest on	

CCP: Coal Combustion Products;

Spillway Size: Pipe Dia. for drop inlet; open channel width (typically emergency or (auxiliary) spillway) at the control section, Ft;.

Freeboard: vertical distance from the emergency spillway control section to the lowest point of the crest of the dam.

## DAM ASSESSMENT FORM



C BOWNSTREAM SLOPE GOOD ACCEPTABLE DEFICIENT DOOR	Problems Noted: ☐ None ☐ Livestock Damage ☐ Erosion, Gullies ☐ Cracks ☐ Sinkholes ☐ Appears Too Steep ☒ Depression or Bulges ☐ Slide ☐ Soft Areas ☐ Trees, Bushes, Briars ☐ Animal Burrows ☒ Other  Comments Trench backfill has settled from previous siphon location at top of slope on east embankment.
D SEEPAGE GOOD  ACCEPTABLE  DEFICIENT  POOR	Problems Noted: ☐ None ☐ Saturated Embankment Area ☐ Seepage Exits on Embankment ☐ Seepage Exits at Point Source ☐ Seepage Area at Toe ☐ Flow Adjacent to Outlet  If Seepage: ☐ Clear ☐ Muddy  Drain Outfalls Seen: Yes ☐ No ☐ Flow: ☐ Clear ☐ Muddy ☐ Dry ☐ Obstructed  Comments: Minor amount of red seepage noted at north abutment.  Wet area noted at toe of east slope, no method of monitoring flow observed.
E PRINCIPAL SPILLWAY GOOD   ACCEPTABLE  DEFICIENT  POOR  □	Description: Drop Inlet, not currently in use.  Problems Noted: ☑ None ☐ Deterioration ☐ Separation ☐ Cracking ☐ Inlet, Outlet Deficiency ☐ Stilling Basin Inadequacies ☐ Trash Rack ☐ Other Comments:
F AUXILIARY SPILLWAY GOOD  ACCEPTABLE  DEFICIENT  POOR	Description: Open Channel  Problems Noted: ☐ None ☐ No Auxiliary Spillway Found ☐ Erosion with Backcutting ☐ Crack with Displacement ☐ Appears to be Structurally Inadequate ☐ Appears too Small ☐ Inadequate Freeboard ☑ Flow Obstructed ☐ Concreted Deteriorated/Undermined ☐ Other  Comments: Temporarily blocked with access road while facility off-line, remove obstruction prior to placing facility back on line.
G MAINTENANCE AND REPAIRS  GOOD  ACCEPTABLE  DEFICIENT  POOR	Problems Noted: ☐ None ☐ Access Road Needs Maintenance ☐ Cattle Damage ☐ Spillway Obstruction ☐ Vegetation on Upstream Slope, Crest, Downstream Slope, Toe ☐ Trees on Upstream Slope, Crest, Downstream Slope, Toe ☐ Rodent Activity on Upstream Slope, Crest, Downstream Slope, Toe ☐ Deteriorated Concrete —Facing, Outlet, Spillway ☐ Gate and/or Drawdown Need Repair ☐ Other  Comments: See comment category F.
H IMPOUNDMENT AREA  GOOD  ACCEPTABLE  DEFICIENT POOR	Problems Noted: None Exposed Ash Ponded Water within Ash Ash blocking spill way Signs of damage from dredging Ash deposits in spillway Other Impoundment receives surface water runoff in addition to sluiced ash: Yes No Release of ponded water could cause overtopping of dam: Yes No N/A Comments: Facility currently off-line. Ash currently covered with burlap.
SATISFACTORY FAIR CONDITIONALLY POOR UNSATISFACTORY	

If this rating is different than the previous inspection, please attach an explanation and reasons for change on page 5.

#### DAM ASSESSMENT FORM



#### Summary of Findings and Recommendations in Attached Table

This visual dam assessment was conducted to assess the general overall condition of the reservoir/ash pond/dam, identify visible deficiencies, and recommend areas for monitoring, additional investigative studies and corrective actions. The assessment is based only on visible features/areas of the dam on the day of inspection; it does not constitute a formal safety inspection nor a review or evaluation from each specialist of an inspection team, such as geologists, civil, geotechnical, structural, or hydraulics engineer. The owner should verify the findings of this report and take corrective actions. This assessment does not relieve the owner/operator from their responsibility to conduct routine inspections, maintenance, repairs, modifications, monitoring, documentation, and/or investigative studies.

Professional Engineer's Sig	nature: Marthallach	Date: <u>(-26-0</u> )
Reviewed by:	Owner/Owner Representative Signature	Date:



## GUIDELINES FOR DETERMINING CONDITIONS

Conditions Observed - Ap		lope, Crest, l and Impoun			Spillwa	y , Auxiliary Spillway
Good In general, this part of the structure has a good appearance and conditions observed in this area do not appear to threater the safety of the dam	is maintained, surfa irregular, eroded, ru	aces may be atted, spalled, bt in new ions in this tly appear to		deterioration and/or ding may threaten the dam.	Poor Conditions observed in this area appear to threaten the safety of the dam. Conditions observed in this area are unacceptable.	
	Condition	ns Observed	- Applies to	Seepage		
Good  No evidence of uncontrolled seepage. No unexplained increase in flows from designed drains. All seepage is clear Seepage conditions do no appear to threaten the safety of the dam.	d other than drain other designed unexplained increa t from designed	outfalls, or drains. No use in flows drains. All ur. Seepage ed do not	areas other and other Seepage nee increase floo deterioration	may threaten the	obser safety unacc Desig have in res seepa 3) conce pondi	sive seepage conditions wed appear to threaten the of the dam and is eptable, Examples: 1) med drain or seepage flow increased without increase servoir level. 2) Drain or ge flows contain sediment. Widespread seepage, entrated seepage or ng appears to threaten the of the dam.
	Conditions Obser	rved – Appli	es to Mainte	nance and Repair	Salety	of the dam.
Good  Dam appears to receive effective on-going maintenance and repair, and only a few mino items may need to be addressed	Acceptable  Dam appears to receive maintenance, but some maintenance items need to be addressed. No major repairs are required.		Deficient Level of maintenance of the dam needs significant improvement. Major repairs may be required. Continued neglect of maintenance may threaten the safety of the dam.		Poor Dam does not receive adequate maintenance. One or more item needing maintenance or repail have begun to threaten the safety of the dam. Level of maintenance is unacceptable.	
	10	Overall C	Conditions			
dam safety deficiencies def recognized. Safe rec performance is expected loa under all anticipated loading conditions, including such events as wo	ir existing dam safety iciencies are ognized for normal ding conditions, requent hydrologic /or seismic events uld probably result in am safety deficiency.	for unusu conditions realistically the expected structure, designation used when exist as analysis which in	sial safety s recognized al loading which may occur during d life of the This may also be uncertainties to critical parameters dentify a dam safety further ns and	recognized for n loading cond Immediate action resolve the defic	clearly normal itions. is to ciency ended; is may until	Unsatisfactory A dam safety deficiency exists for normal conditions. Immediate remedial action is required for problem resolution.

# Findings and Recommendations

Plant Name: E.W. Brown

Structure Name: Main Ash Pond

State Facility ID: 737

Assessment date: 1/19/2009

Note: Pond currently not on-line and is not ponding water.

	sprinkler line	narrowing	Excavate and fill depressions at downstream slope at previous drawdown pipe location	onitoring of flow	Monitor flow for changes, evaluate whether seepage source is from cooling tower discharge	Remove blockage in Emergency Spillway prior to placing facility back on-line	Prepare Operation and Maintenance Plan for all aspects of structure	Prepare Emergency Action Plan (EAP) for structure distress scenarios	Institute and document regular facility inspection plan	Conduct visual inspection of facility during 2009 growing season	raphic mapping		
Description	Fill depression under sprinkler line	Repair upstream crest narrowing	Excavate and fill depre	Install weir to allow monitoring of flow	Monitor flow for chang	Remove blockage in E	Prepare Operation an	Prepare Emergency A	Institute and documer	Conduct visual inspec	Prepare current topographic mapping		
Location Description	Crest	Crest	Crest	Toe	North Abutment	Spillway	General	General	General	General	General		
Photo Number	3	4	5	12	13	8	N/A	N/A	N/A	N/A	N/A		
Priority Rating	Moderate	Normal	Moderate	Normal	Moderate	Moderate	Normal	Moderate	Normal	Moderate	Normal		
Item Number	1	2	3	4	5	9	1	8	6	10	11		

High - Recommend that action item be addressed as soon as possible. Priority:

Moderate - Recommend that action item be addressed as soon as feasible - preferably before the next state inspection.

Normal - Recommend that action item be addressed as part of the ongoing maintenace of the structure.

Location:

Abutment Crest

Principal Spillway Emergency Spillway Toe

Downstream Slope

Upstream Slope



Photo #1: Upstream slope, East embankment, looking South

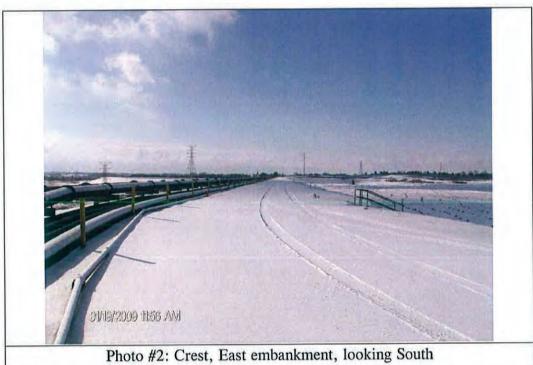




Photo #3: Downstream side of crest, East embankment, small depression below pipe, looking South



Photo #4: Narrowing Crest, Upstream slope, East embankment, looking South

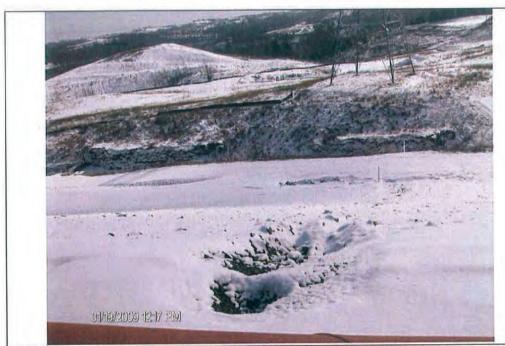


Photo #5: Depression up to 5' wide and 1.5' deep, Downstream slope, East embankment, looking South

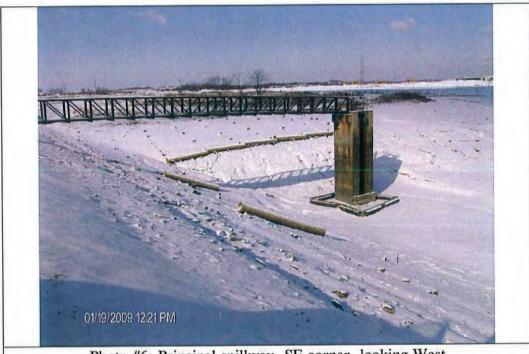


Photo #6: Principal spillway, SE corner, looking West

## E. W. BROWN MAIN ASH POND

January 19, 2009

Photo #7: Erosion gully in ash, upstream slope next to principal spillway, SE corner, looking SW



Photo #8: Earthen berm blocking emergency spillway, SE corner, looking South



Photo #9: Emergency spillway, SE corner, looking North toward pond

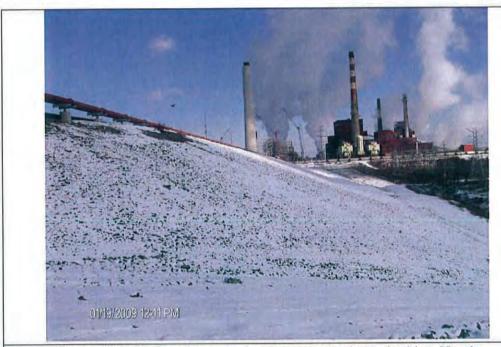


Photo #10: East embankment, downstream slope, looking North

## E. W. BROWN MAIN ASH POND

January 19, 2009

01/19/2009 tl/59 AM

Photo #11: Narrowing crest, just South of NE corner, looking NW



Photo #12: Water flowing, East embankment at toe, looking East toward Herrington Lake

## E. W. BROWN MAIN ASH POND

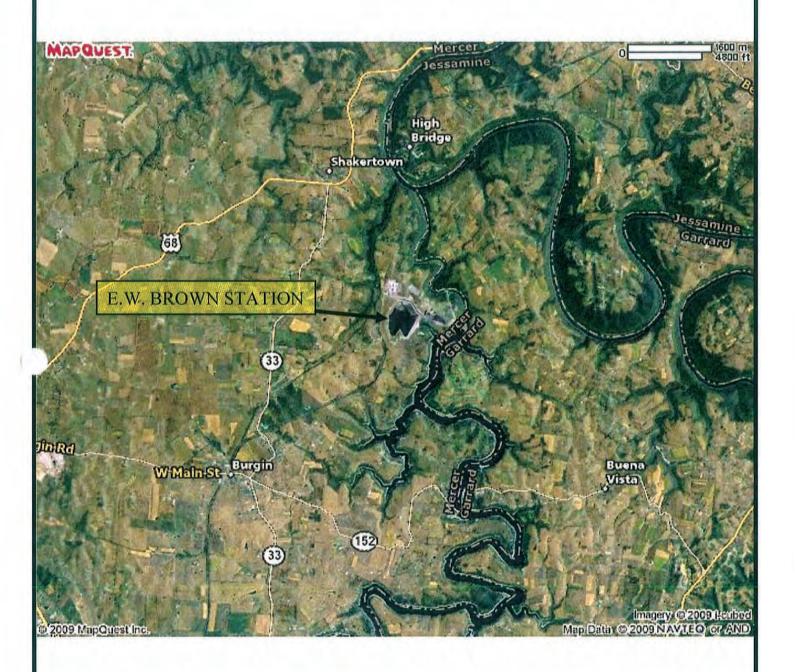
January 19, 2009

01/19/2009 OHS PM

Photo #13: "Red" Water, possibly from cooling tower discharge, North abutment, downstream slope, looking North



Photo #14: East embankment, downstream slope, looking NE





132 Citizens Boulevard Simpsonville, KY 40067 (502) 722-1401

PROJECT NO: 27.11000.9G99

DESIGNED BY: RR SCALE:N/A REVIEWED BY: JE
DRAWN BY: RR DATE: 1/27/09 FIGURE: 1

#### SITE VICINITY MAP

E.W. BROWN MAIN ASH POND EON-US Dam Inspections Burgin, KY



2009 POND ASSESSMENT SITE PLAN E.W. BROWN MAIN ASH POND KU/E.ON U.S.

BURGIN, KY

LEGEND:

LOCATION OF PHOTOGRAPHY — DIRECTION OF PHOTOGRAPHY — PHOTO DESIGNATION

Project Humber: 27,11000.9G99

Oresing Fac: E.ON Dom Inspections

Chd. Oys

JE





# STATE FILE REVIEW INFORMATION WORK SHEET

SHE:	E w Brown - Main As	n Pond
ID#:	737	
HAZARI	RATING:A (low)	
COPY O	F RATING CERTIFICATI	ON: Referenced in file
RECOM	MENDED INSPECTION	FREQUENCY:
DATE OF	FLAST INSPECTION:	11/4/04
DATES C	F PREVIOUS INSPECT	IONS:
10/22/98		5/3/90 Constr Insp of Mod.
1/04/96 Ha	zard Rating - Low	9/17/87 Hazard Rating - Moderat
2/6/92	AND THE RESERVE	3/30/83
10/29/90 C	onstr. Insp of Modification	6/14/77

#### INSPECTION FINDINGS (deficiencies):

11/04 - Potholes on crest, erosion near spillway inlet

2/92- Monitor seepage for change in color or volume

10/90 - Significant seepage during construction of modification

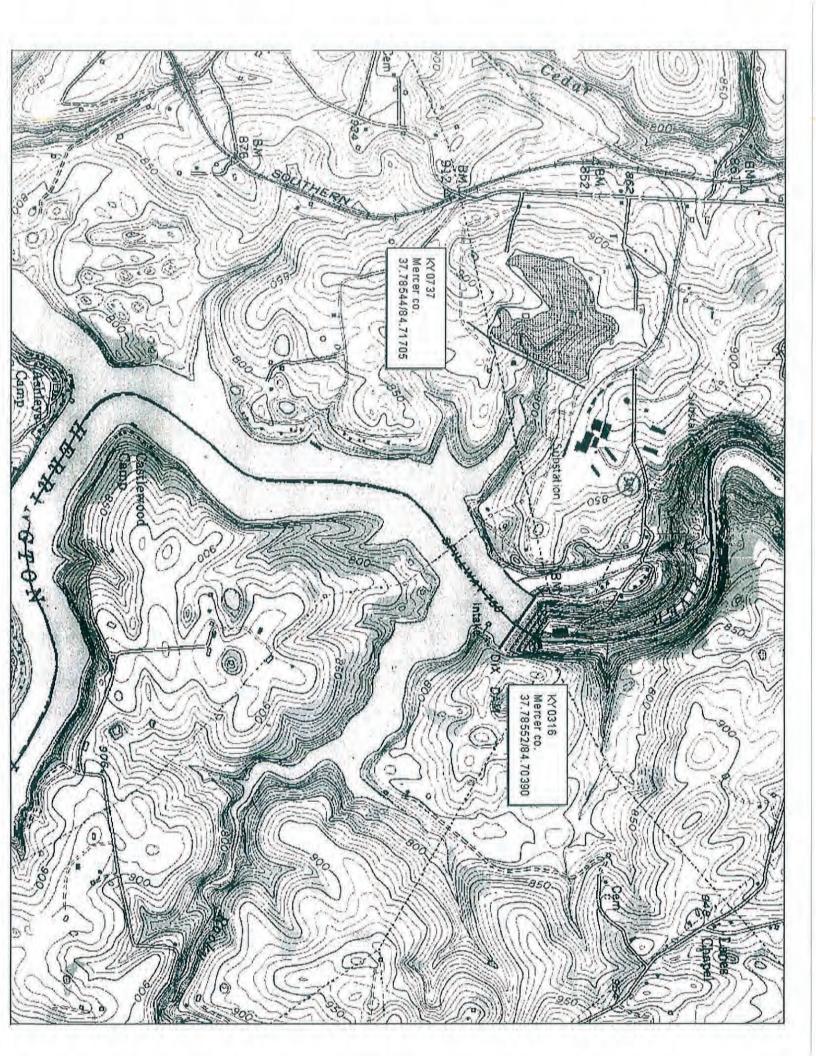
9/87 - Trees and bushes on downstream slope

3/83 - Mow regularly, cut trees and bushes, monitor seepage

9/77 - general maintenance regarding vegetation

OTHER INFORMATION AVAILABLE (design criteria, modifications, etc): Phase II & III Investigation Report (design reports) 1973 H.C. Nutting

Date:	1/22/09				
By:	DHB				
Additio	nal Sheets:	6 copies	from	DOW	files





ERNIE FLETCHER
GOVERNOR

## ENVIRONMENTAL AND PUBLIC PROTECTION CABINET

LAJUANA S. WILCHER SECRETARY

DEPARTMENT FOR ENVIRONMENTAL PROTECTION
DIVISION OF WATER
14 REILLY ROAD
FRANKFORT, KENTUCKY 40601-1190
www.kentucky.gov
November 4, 2004

KY Utilities Co E.W. Brown Generating Station 815 Dix Dam Road Harrodsburg, KY 40330

Re: Scheduled Inspection ID of Dam: 0737

KENTUCKY UTILITIES FLY ASH DAM (2)

Mercer County, KY Hazard Class: LOW

Dear KY Utilities Co:

On November 3, 2004, personnel from the Natural Resources and Environmental Protection Cabinet, Division of Water, inspected the above referenced structure. A copy of the inspection report is enclosed. The Division of Water is responsible for performing safety inspections of dams in Kentucky.

Kentucky Revised Statutes Chapter 151 (KRS 151) and associated regulations establish minimum maintenance and design criteria for dams. KRS 151.125 gives the Division of Water authority to require any measures necessary to bring the dam into compliance with statutes and regulations. As the owner you are required to maintain the dam to assure public safety.

Based on our visual inspection of the dam, the following notes/deficiencies were noted:

- 1. Structure is well maintained.
- 2. Potholes on the crest of the dam must be filled.
- 3. Erosion near the spillway inlet should be monitored and repaired.

If you have any questions concerning this matter, please contact Scott Phelps at (502) 564-3410.

Sincerely,

Ron Dutta, P.E., Supervisor

Dam Safety and Floodplain Compliance Section

Water Resources Branch

Division of Water

Enclosure:

# COMMONWEALTH OF KENTUCKY DEPARTMENT FOR ENVIRONMENTAL PROTECTION DIVISION OF WATER 14 REILLY ROAD FRANKFORT, KENTUCKY 40601

# CERTIFICATE OF INSPECTION FOR DAM AND APPURTENANT WORKS

Note: The Division of Water does not intend this report to be taken as an assurance that no other problems exist at this site or that this dam is safe. The reports sole intent is to provide you a factual account of the conditions observed at the site during the inspection. If you have questions, write this office at the above listed address or call (502) 564-3410.

ID of Dam:

Weather:

0737

Name of Dam:

KENTUCKY UTILITIES

FLY ASH POND No. 2

М

County: Inspection Date: Mercer

November 3, 2004

November 3, 200

55° F, Overcast

Inspection Type: Dams

Hazard Class: LOW

Owner: KY Utilities Co

EW Brown Generating Station

Address:

815 Dix Dam Road Harrodsburg

City: State:

KY

Zip:

40330

Phone:

859-748-4401

Persons Present at Inspection: Scott Phelps, Marilyn Thomas, Ramendra Dutta, and Thomas Moore of KU

Height of Dam:

Type of Dam:

126 feet

Latitude Dec Deg: 3 Longitude Dec Deg: -

37.786389 -84.716944 Normal Pool Elevation (MSL): 868
Current Pool Elevation (MSL): 868
Eman Spillman Elevation (MSL): 8

-84.716944 Emer. Spillway Elevation (MSL): NA
Earthfill & Rockfill embankment, 2175 ft long with a top width of 20 ft. A 5' berm is on downstream

side. Side slopes are both 2:1.

Upstream Slope of Dam: Rock cover without vegetation. No signs of major crosion. Minor crosion found near the spillway inlet. No signs of slumps, slides, subsidence, or cracking.

Crest of Dam: Gravel road without vegetation. No signs of cracking or subsidence. Few potholes were noted in the crest.

Downstream Slope of Dam: Rock cover without any vegetation. No signs of animal burrows, slides, slumps, or cracking.

Toe Drains: NA

### CERTIFICATE OF INSPECTION FOR

#### KY ID: 0737

Principal Spillway: Existing 24" diameter drop inlet. No outlet works indicated for new structure on plans supplied by owner.

Principal Spillway Comment: Catwalk extending out to inlet but no signs of problems. Valve allowing water out was closed at time of visit so floats can be cleaned/repaired.

Stilling Basin: Good condition. Some ash noted behind the weir.

Emergency Spillway: No spillway indicated on plans for the revised structure. Water is pumped into structure & pumped out to control pool level.

**Emergency Spillway Comments: NA** 

Drawdown System: NA

Location of Drawdown Valve: None Last Date of Operation: NA

Does Hazard Classification need to be reevaluated? No changes in downstream conditions.

Were Photographs Taken? Yes

General Comments and Recommendations:

Structure is in excellent condition. Potholes in crest should be filled. Erosion near spillway should be monitored and repaired as necessary.

Inspector: Scott Phelps

Reviewer: Ron Dutta Date: 11/5/2004

### MEMORANDUM

TO: K.U. Ash Pond, No. KY0737

THRU: George A. Childers, P.E. AAC

Dam Safety and Floodplain Compliance Section

FROM: James W. Marchant, P.E.

Dam Safety and Floodplain Compliance Section

DATE: October 31, 1990

SUBJECT: Inspection of Fly Ash Pond at K.U.'s E.W. Brown

Generating Station, Mercer County

#### Introduction

On October 29, 1990, G.A. Childers and J.W. Marchant inspected the subject dam. Craig Avery (F.M.S.M), Mark Caudill (F.M.S.M.), Charles Matherly (K.U.) and Bernadette Bay (K.U.) were also present at the inspection.

Work at the dam is substantially completed. Some final grading of the slopes near the right end of the dam is required. The crest road is about 50% complete. Mr. Avery believes all work will be completed in about a week. Asconstructed drawings will be completed and submitted to D.O.W.

Mr. Avery said they asked us to perform this inspection before the contractor moved from the site in case additional work is necessary. We noted that we did not observe any omissions in the work during our inspection.

#### Seepage

said heavy There are significant foundation seepage flows. Mr. Avery flows were encountered during construction. Flow through the rock drain The upper beneath the downstream shell is routed to three discharge points. right and upper left drain discharge is routed through 8-inch plastic drain The pipes discharge to open drain channels. Flow in the right and left drain was about 50 and 20 gpm, respectively. Most flow emerged through the rock drain at the toe of the dam. Flow was seeping through the gravels to an elevation up to 5 feet above the toe. Mr. Avery noted that the gravel and rock drain system was designed to handle these flows. Flow was estimated to be about 200 to 300 gpm. We discussed the need for a flow measuring weir in the toe area. Mr. Avery said he had no plans to install a weir. We agreed to visually monitor the flow for now. If changes in flow rates are noted, installation of a weir should be considered.

MRMORANDUM K.U. Fly Ash Pond, No. KY0737 October 31, 1990 Page 2

## Instrumentation

Pneumatic piezometers and settlement gages were built into the embankment. Gages are remotely monitored at two locations along the toe of the dam. There are about 90 piezometers and 4 settlement gages. Mr. Avery said that gages were monitored weekly and sometimes daily during construction. Now they are being read monthly. I requested an annual instrumentation report to be submitted by the owner or their engineer. The report should contain tabulated gage readings, and an evaluation of the dams performance based on the gage readings. In the submittal of an instrumentation report, the following items should also be considered:

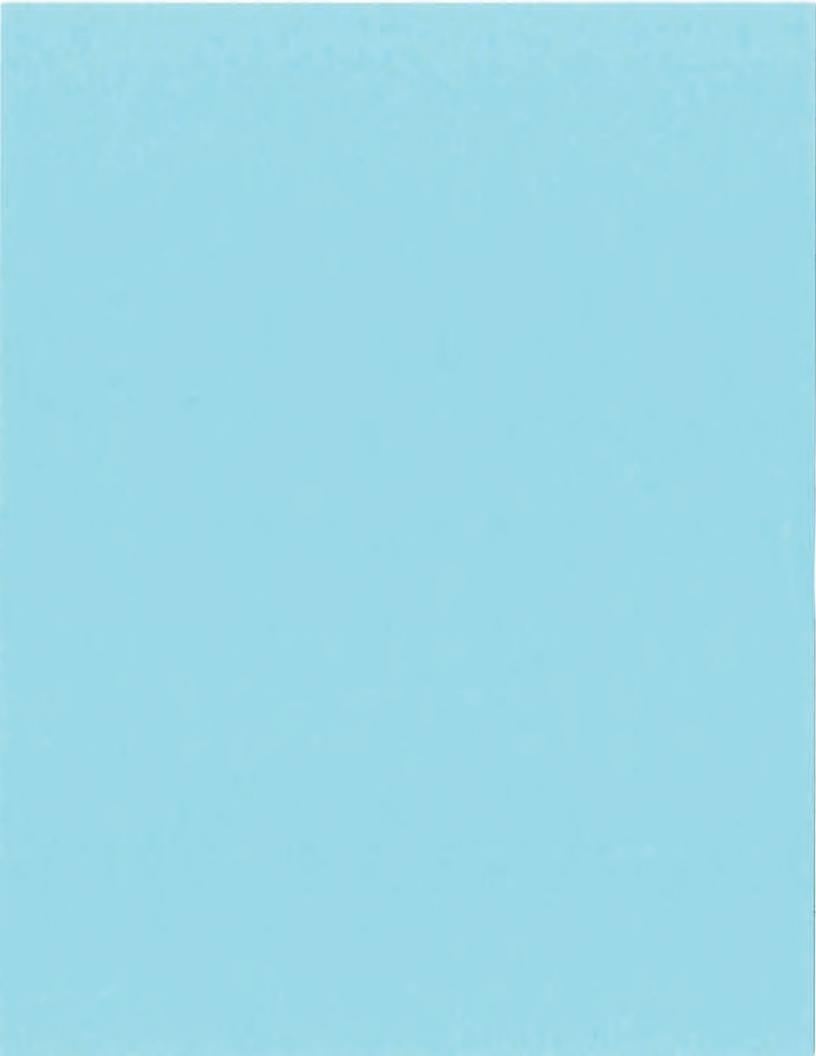
Present the data graphically.

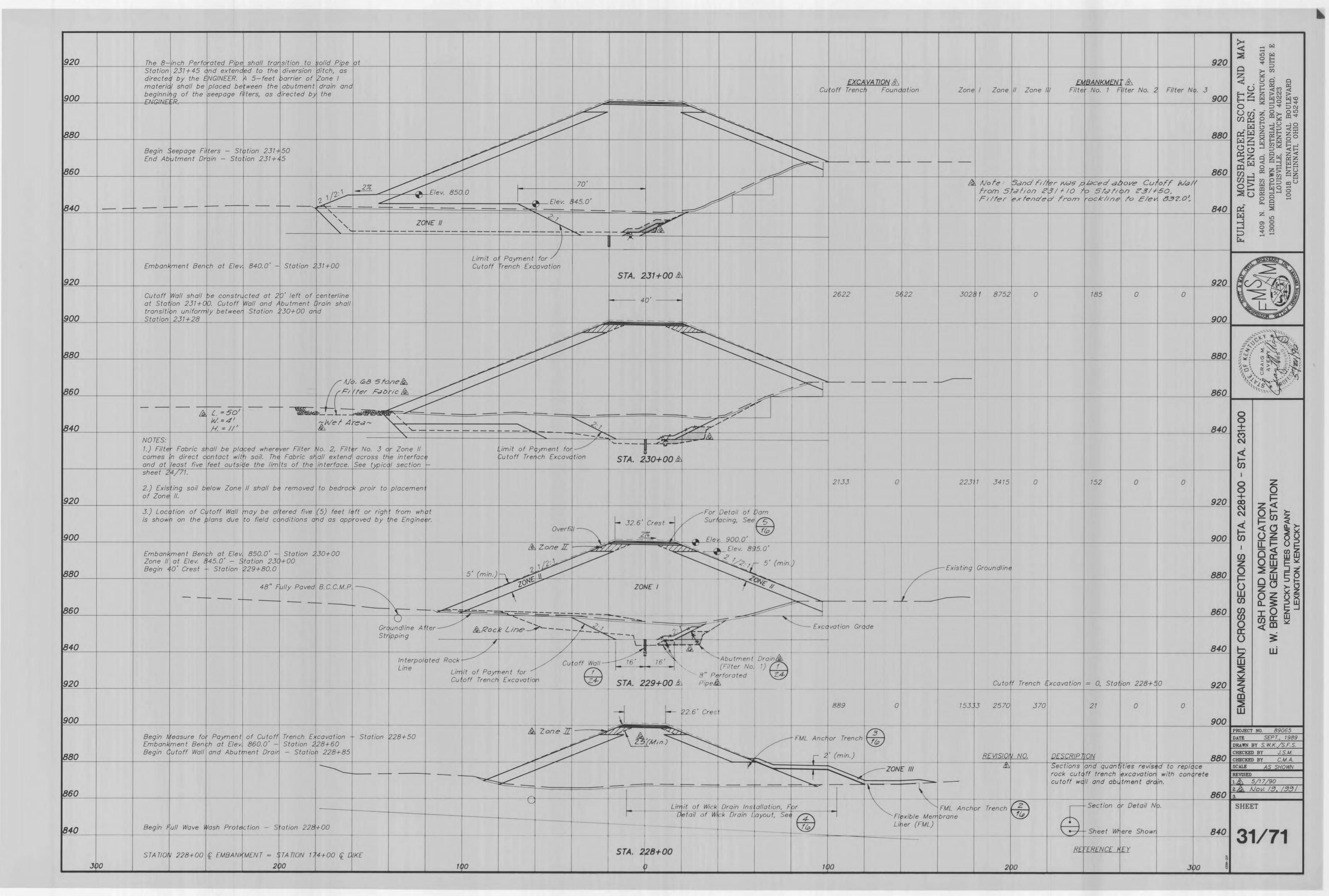
Keep a record of reservoir elevation and show on graphs.

Supply data in computer format such as EXCEL or LOTUS123 if possible.

## Principal Spillway Structure

The new outlet is in operation. Reservoir level is controlled by adding or removing stoplogs. Stoplogs can be installed in 6-inch increments. A skimmer prevents floating debris from being released. When the embankment grading is complete, the access bridge for the principal spillway structure will be set in place.





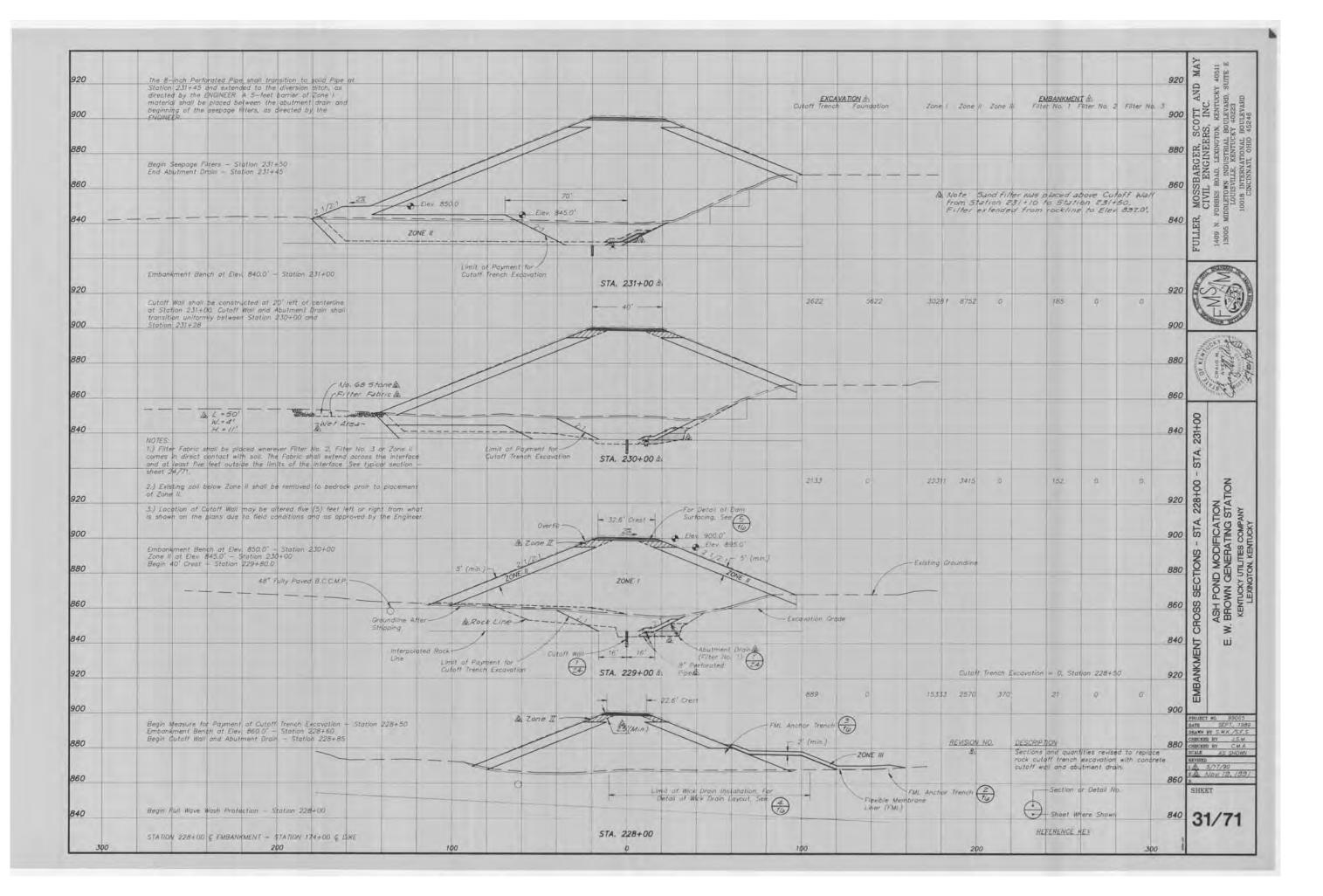




Photo 1: Main Pond: Downstream Slope at Southeast Section

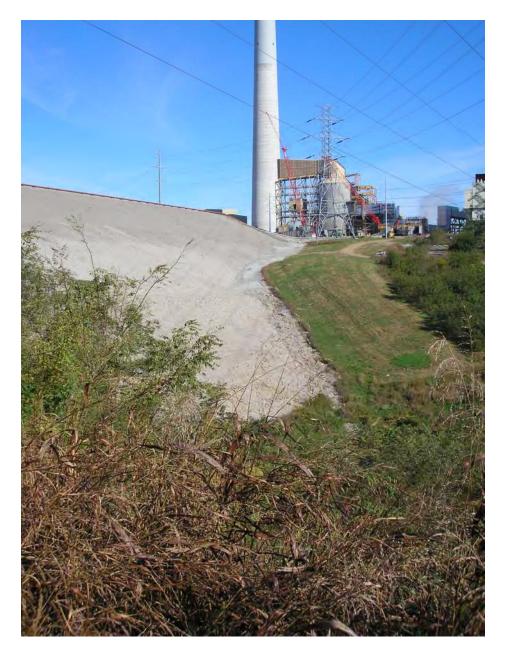


Photo 2: Main Pond: Downstream Slope at Southeast Section



Photo 3: Main Pond: Downstream Slope at Southeast Section



Photo 4: Main Pond: Toe at Southeast Section



Photo 5: Main Pond: Downstream Slope from Toe at Southeast Section

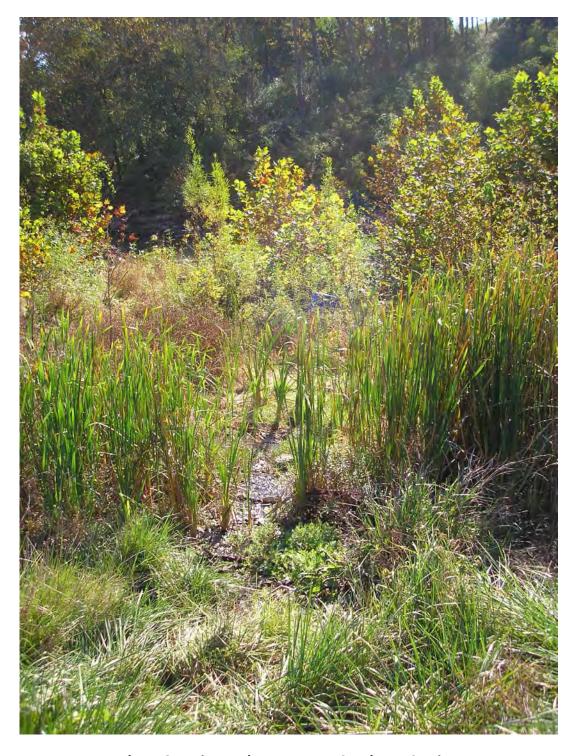


Photo 6: Main Pond: Toe Area at Southeast Section

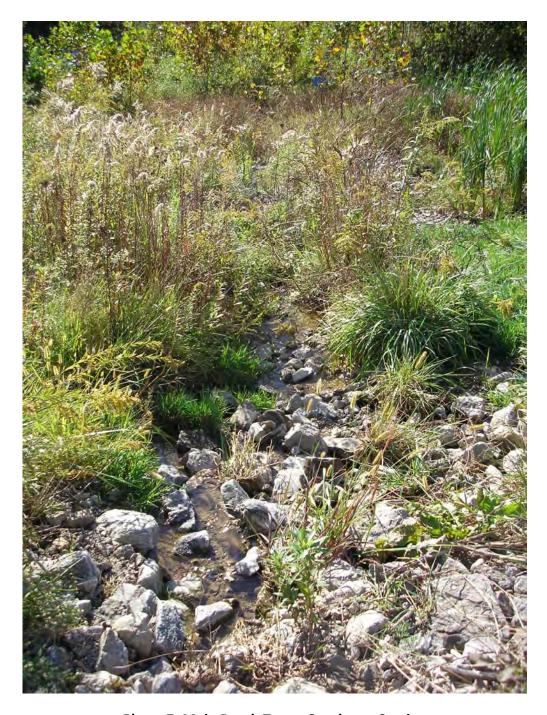


Photo 7: Main Pond: Toe at Southeast Section



Photo 8: Main Pond: Downstream Slope from Toe at Southeast Section



Photo 9: Main Pond: Downstream Slope from Toe at Southeast Section



Photo 10: Main Pond: Toe Area at Southeast Section



Photo 11: Main Pond: Downstream Slope from Toe at Southeast Section



Photo 12: Main Pond: Downstream Slope from Toe at Southeast Section



Photo 13: Main Pond: Downstream Slope. Bench Road at Elev. 870 Crest



Photo 14: Main Pond: Downstream Slope. Bench Road at Elev. 870 Crest

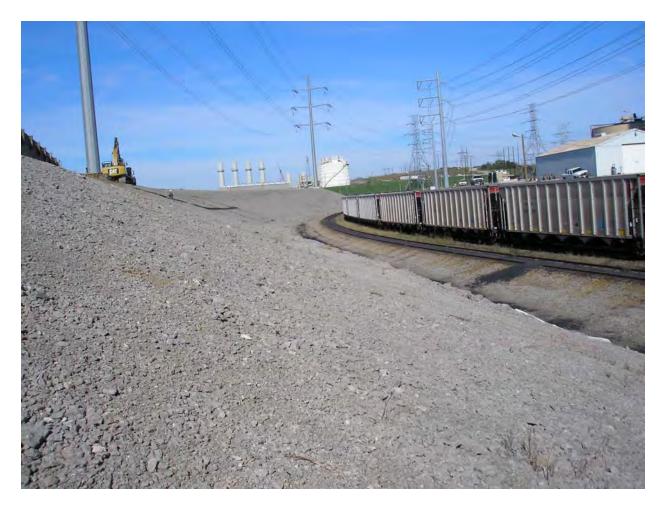


Photo 15: Main Pond: Downstream Slope at North Section



Photo 16: Main Pond: Upstream Slope at North Section



Photo 17: Main Pond: Upstream Slope at North Section



Photo 17: Main Pond: Upstream Slope at Southeast Section



**Photo 19: Main Pond at Northeast Area** 



Photo 20: Main Pond at Northeast Area



**Photo 21: Main Pond: Crest of Southeast Section Facing Southwest** 



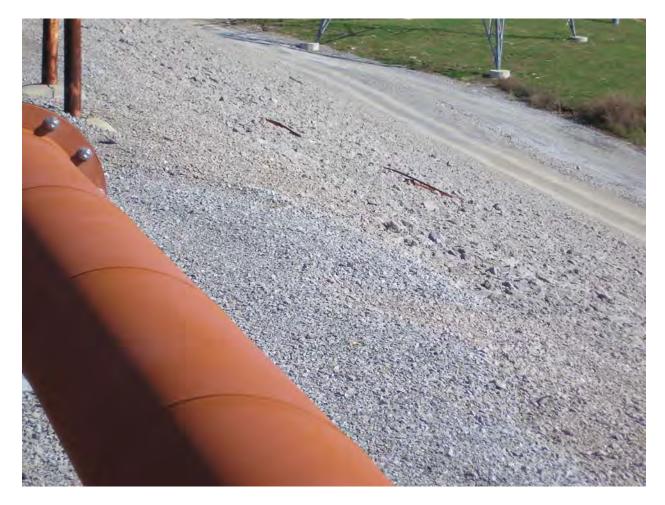
Photo 22: Main Pond Facing North from Southeast Embankment



Photo 23: Main Pond Facing North from Southeast Embankment



Photo 24: Main Pond Facing North from Southeast Embankment



P-00309: Main Pond: Downstream Slope at Southeast Section



Photo 26: Main Pond: Downstream Slope at Southeast Section



Photo 27: Main Pond: Downstream Slope at Southeast Section



Photo 28: Main Pond: Existing Decant Spillway



Photo 29: Main Pond, Southern End of Pond



Photo 30: Main Pond: Crest and Upstream Slope – Southeast Section Facing Northeast



Photo 31: Main Pond: Existing Decant Spillway. New Spillway and Access Bridge in Background



Photo 32: Main Pond: Existing Decant Spillway



Photo 33: Main Pond: Existing Decant Spillway



Photo 34: Main Pond: Crest and CCW Pipes along Southeast Section



Photo 35: Main Pond: Crest and CCW Pipes into Auxiliary Pond at South End of Main Pond

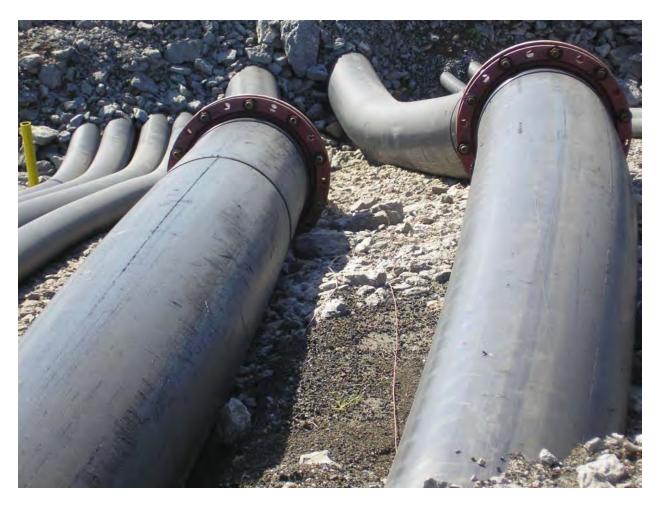


Photo 36: Main Pond: Crest and CCW Pipes into Auxiliary Pond at South End of Main Pond

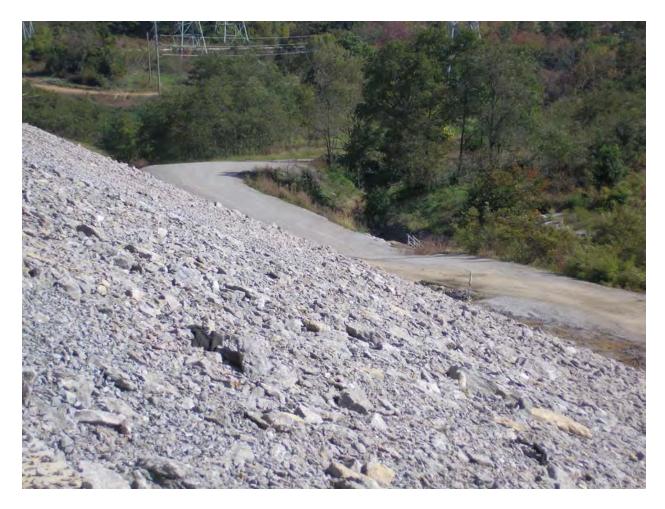


Photo 37: Main Pond: Downstream Slope at South End of Southeaster Section



Photo 38: Main Pond: Downstream Slope at South End of Southeaster Section



Photo 39: Main Pond: Crest and Upstream Slope – Southeast Section Facing Northeast



Photo 40: Main Pond: Existing Decant Spillway



Photo 41: Main Pond: Existing Decant Spillway



Photo 42: Main Pond: Existing Decant Spillway



Photo 43: Main Pond: Crest Near Southeastern Abutment: Construction of New Decant Spillway



Photo 44: Main Pond: Downstream Slope at South End of Southeaster Section



Photo 45: Main Pond: Crest Near Southeastern Abutment: Construction of New Decant Spillway



Photo 46: Main Pond: Crest Near Southeastern Abutment: Construction of New Decant Spillway



Photo 47: Main Pond: Crest and Upstream Slope – Southeast Section Facing Northeast



Site Name: E. W. Brown Ge. Juit Name: Mayor Fla Ash	_ ,		Operator's Name: Kentucky Ut.	lifies	Co.
Unit I.D.: Ky 73				Significar	
nspector's Name: Hugh A. Wan	10z	(V	7164) Toscoh P. Klein II		
eck the appropriate box below. Frovide comments wh	en approi	priate. I	f not applicable or not available, record "N/A". Any unusual	conditions	s or
istruction practices that should be noted in the comme bankment areas. If separate forms are used, identify a	nts sectio pproxima	n. For tea	arge diked embankments, separate checklists may be used that the form applies to in comments.	for differe	ent_
	Yes	No		Yes	No
Frequency of Company's Dam Inspections?	Wee	14	18. Sloughing or bulging on slopes?		1
Pool elevation (operator records)?	*	/	19. Major erosion or slope deterioration?		1
Decant inlet elevation (operator records)?	86	3.0	20. Decant Pipes:		
Open channel spillway elevation (operator records)?			is water entering inlet, but not exiting outlet?		V
Lowest dam crest elevation (operator records)?	900	1.0	Is water exiting outlet, but not entering inlet?		1
If instrumentation is present, are readings recorded (operator records)?	1		Is water exiting outlet flowing clear?	WA	Vic
Is the embankment currently under construction?		1	21. Seepage (specify location, if seepage carries fines, and approximate seepage rate below):		
Foundation preparation (remove vegetation, stumps, psoil in area where embankment fill will be placed)?	1		From underdrain?		V
Trees growing on embankment? (If so, indicate largest diameter below)		1	At isolated points on embankment slopes?		1
Cracks or scarps on crest?		1	At natural hillside in the embankment area?		1
Is there significant settlement along the crest?		/	Over widespread areas?		V
. Are decant trashracks clear and in place?		*	From downstream foundation area?		1
Depressions or sinkholes in tailings surface or whirlpool in the pool area?		1	"Boils" beneath stream or ponded water?		V
Clogged spillways, groin or diversion ditches?		*	Around the outside of the decant pipe?	-	V
Are spillway or ditch linings deteriorated?		*	22. Surface movements in valley bottom or on hillside?		1
Are outlets of decant or underdrains blocked?		7	23. Water against downstream toe?		1
Cracks or scarps on slopes?	17	1	24 Were Photos taken during the dam inspection?		
ajor adverse changes in these items cou rther evaluation. Adverse conditions no llume, etc.) in the space below and on th	ted in t	hese i	tems should normally be described (extent, I	ocation	),
spection Issue #	Comm	nents			
'z ^^	10.15	11.	ash pond has been dewater.	./.	
					1 <i>9</i>
	cur	rent	by out of service for expans	100.	
7\$8 50	- /	1.	preparation underway to		//
1 T O	/				./ 🛩

Existing decent viser is out of service. New riser Spillways and outlets are out of service during expansion construction.

Water is neither entering or exiting out of service inlet or outlet **EPA FORM -XXXX** #20-

## **U. S. Environmental Protection Agency**

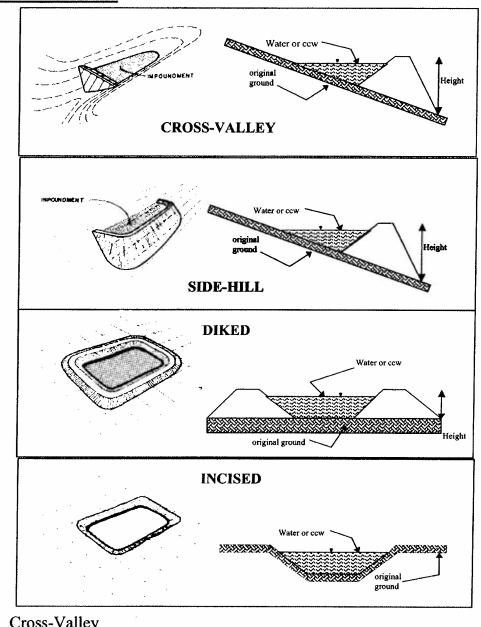


## Coal Combustion Waste (CCW) Impoundment Inspection

		Hugh. A. Ward, P.E.
Impoundment NPDE	S Permit # <u> </u>	
Date Feb 1, 200	S Permit # <u>KY 000 Z0 Z0</u> 22 - Jan 31, Z007	,
Impoundment Nan Impoundment Con EPA Region State Agency (Fiel Name of Impounds	ne Main Ash Pond, npany Kentucky Utilities	Company  To Environmental Profession  14 Reilly Rd., Frankfurt, Ky 4060.
NewUp	odate 🖊	
<del>-</del>	urrently under construction? Trently being pumped into	Yes No
IMPOUNDMENT	FUNCTION: Fond is current been dewaters	ntly out of sevice and hus
Impoundment Location:	m Town: Name High Bridge mpoundment Z. 5 m. les  Longitude 34 Degrees 43  Latitude 37 Degrees 47  State KY County Merces	_ Minutes _ 08 Seconds _ Minutes _ 15 Seconds
Does a state agency	regulate this impoundment? YES Agency? Department of Envir	S
II bo winen state F	Ourse of Whater	

following would occur):
LESS THAN LOW HAZARD POTENTIAL: Failure or misoperation of the dam results in no probable loss of human life or economic or environmental losses.
LOW HAZARD POTENTIAL: Dams assigned the low hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.
SIGNIFICANT HAZARD POTENTIAL: Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.
HIGH HAZARD POTENTIAL: Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.
DESCRIBE REASONING FOR HAZARD RATING CHOSEN:  Qualitative assessment of threat to downs freem residences
Dam break analysis and prepura him of Emergancy Action Plan currently underway.

## **CONFIGURATION:**



Closs=valley		
Side-Hill		
Diked		
Incised (form completion optional)		
Combination Incised/Diked		<b>2</b>
Embankment Height for	feet	Embankment Material Rock f.//
Pool Area 75./2 a	icres	Liner Bo m. Il Low Density Polyethylane flex, blo Liner Permeability Tomano le membrone
Current Freeboard _N/A fe	eet	Liner Permeability Imperate membrane

## TYPE OF OUTLET (Mark all that apply)

Open Channel Spillway	TRAPEZOIDAL	TRIANGULAR
Trapezoidal	Top Width	Top Width
Triangular		<b>—</b>
Rectangular	Depth	Depth
Irregular	Bottom Width	
depth	RECTANGULAR	IRREGULAR
bottom (or average) width		Average Width
top width	Depth	Avg Depth
✓_ Outlet		
30" inside diameter		
Material		Inside Diameter
corrugated metal	1	/
welded steel	\	
concrete		
plastic (hdpe, pvc, etc.) other (specify)		•
s water flowing through the outlet?	YES NO	
No Outlet		
Other Type of Outlet (specif	Y) Trapezoida/ Or Top width 24/1	t. Depth: 4ft.
The Impoundment was Designed By  May Enqueers	Fuller, Mossbarg	ver, Scott &
• • • • • • • • • • • • • • • • • • • •		

Has there ever been a failure at this site? YES	NO
If So When?	
If So Please Describe:	

Has there ever been significant seepages at this site? YES	NO
If So When?	
IF So Please Describe:	